6000. STEEL

- 6100 & 6200
  - 6130 - Design Data, Principles and Tools
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6300. Design -

6310. Structural Steel Members and Components

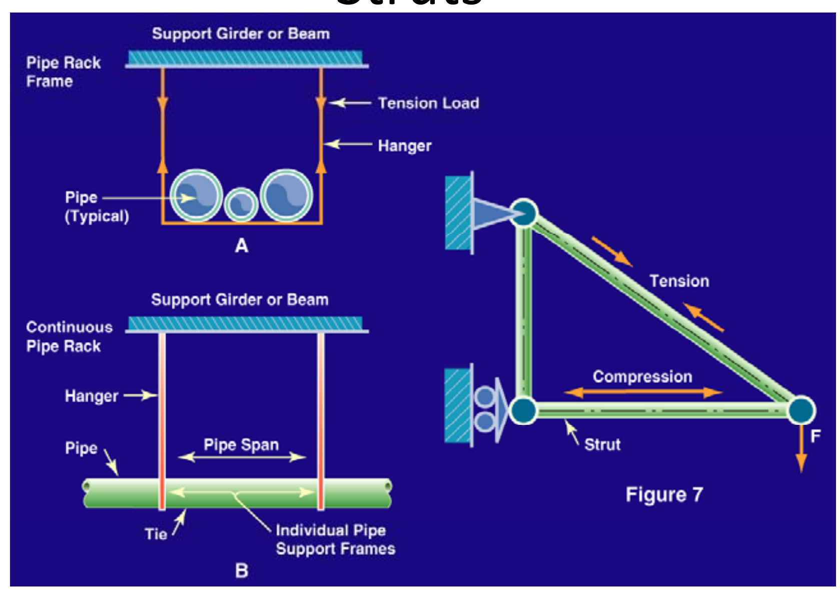
- Module 1: Tension (Sections ND and use of AISC Manual Part 5 – Tension Member Table)
- Module 2: Flexure and Shear (Sections NF and NG and use of AISC Manual Part 3 - Beam Design Table)
- Module 3: Compression (Section NE and use of AISC Manual Part 4 - Column Design Table)
- Module 4: Composite Members (Section NL and use of AISC Manual Composite Beam Design Tables 3-19 & 3-20)

6310. Structural Steel Members and Components – Module 1: Tension

This section of the module covers:

- Introduction
- Design strength
- Net area
- Staggered fasteners
- Block shear
- Design of tension members
- Threaded rods, pin-connected members

Tension Loading : Ties, Hangers, and Struts



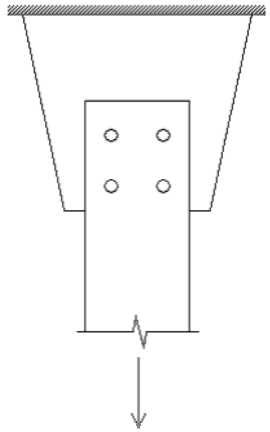
Introduction

- Stresses (f) in axially loaded members are calculated using the equation

$$f = P/A$$

where P is the load and A is the cross-sectional area normal to the load.

- Design of this component involves calculations for
  – Tension member (gross area)
  – Tension member at connection (net area)
  – Gusset plate at connection (net area)
  – Gusset plate at support



Design Strength

A tension member can fail by

- Excessive deformation (yielding) - Excessive deformation is prevented by limiting stresses on the gross section to less than the yield stress. For yielding on the gross section, the nominal strength is:

$T_n = F_y A_g$ and $\varphi_t = 0.90$

- Fracture - Fracture is avoided by limiting stresses on the net section to less than the ultimate tensile strength. For fracture on the net section, the nominal strength is:

$T_n = F_u A_e = F_u (UA_n)$ and $\varphi_t = 0.75$

where A_e is the effective net area, A_n is the net area and U is the reduction coefficient (an efficient factor)

Net Area

Net Area -

The performance of a tension member is often governed by the response of its connections. The AISC Steel Manual introduces a measure of connection performance known as joint efficiency, which is a function of

– Material properties (ductility)
– Fastener spacing
– Stress concentrations
– Shear lag (Most important of the four and addressed specifically by the AISC Steel Manual)

Net Area

The AISC Steel Manual introduces the concept of effective net area to account for shear lag effects.

- For bolted connections: $A_e = UA_n$
- For welded connections: $A_e = UA_g$

where shear lag factor $U = 1 - \bar{x}/L \leq 0.9$

and $\bar{x}$ is the distance from the plane of the connection to the centroid of the connected member and L is the length of the connection in the direction of the load.

Net Area

$\bar{x}$

AISC Steel Manual

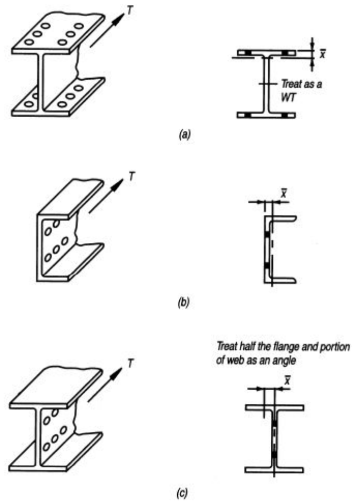


Fig. C-D3.1. Determination of $\bar{x}$ for U.

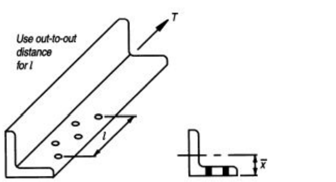
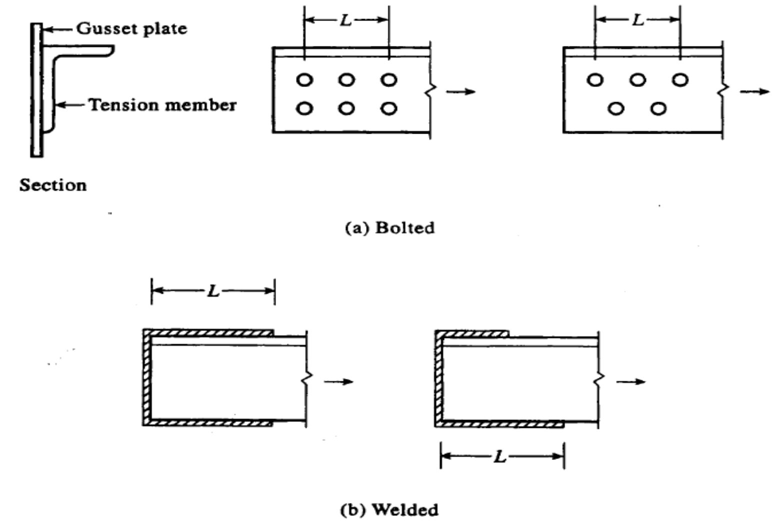


Fig. C-D3.2. Determination of l for U for bolted connections with staggered holes.

Net Area



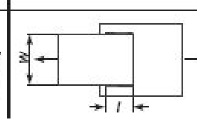
Net Area

- For bolted connections, AISC Table D3.1 gives values for U that can be used in lieu of detailed calculation.

7	W, M, S or HP Shapes or Tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used)	with flange connected with 3 or more fasteners per line in direction of loading	$b_f \geq 2/3d \ldots U = 0.90$ $b_f < 2/3d \ldots U = 0.85$	—
		with web connected with 4 or more fasteners in the direction of loading	$U = 0.70$	—
8	Single angles (If U is calculated per Case 2, the larger value is permitted to be used)	with 4 or more fasteners per line in direction of loading	$U = 0.80$	—
		with 2 or 3 fasteners per line in the direction of loading	$U = 0.60$	—

Net Area

- For welded connections, AISC Table D3.1 lists

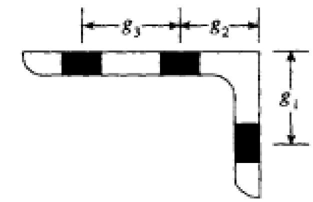
3	All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sectional elements.	$U = 1.0$ and A_n = area of the directly connected elements	—
4	Plates where the tension load is transmitted by longitudinal welds only.	$l \geq 2w \ldots U = 1.0$ $2w > l \geq 1.5w \ldots U = 0.87$ $1.5w > l \geq w \ldots U = 0.75$	

Staggered Fasteners

- Failure line - When a member has staggered bolt holes, a different approach to finding A_e for the fracture limit state is taken. This is because the effective net area is different as the line of fracture changes due to the stagger in the holes. The test for the yielding limit state remains unchanged (the gross area is still the same).
- For calculation of the effective net area, the Section B2 of the AISC Steel Manual makes use of the product of the plate thickness and the net width. The net width is calculated as

$$w_n = w_g - \sum d + \sum \frac{s^2}{4g}$$

Staggered Fasteners



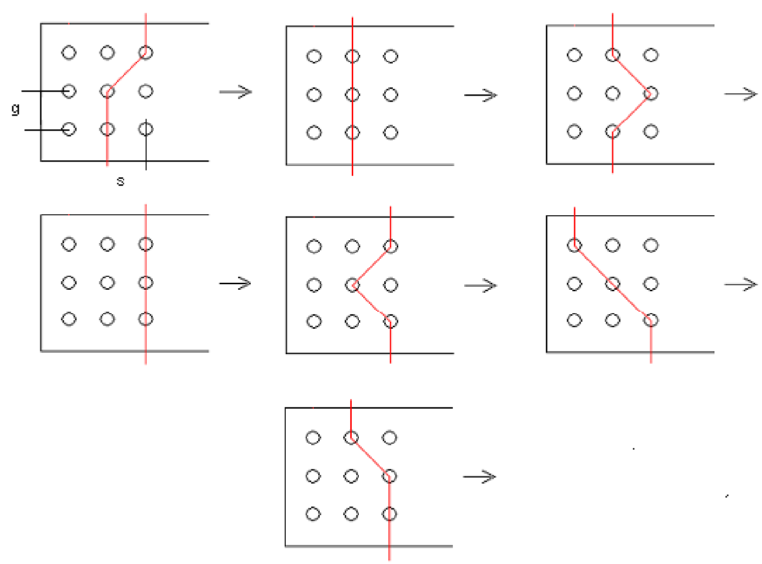
(a)

Usual gages for angles (inches)

Leg	8	7	6	5	4	3½	3	2½	2	1¾	1½	1⅜	1¼	1
g_1	4½	4	3½	3	2½	2	1¾	1⅜	1⅛	1	⅞	⅞	¾	⅝
g_2	3	2½	2¼	2										
g_3	3	3	2½	1¾										

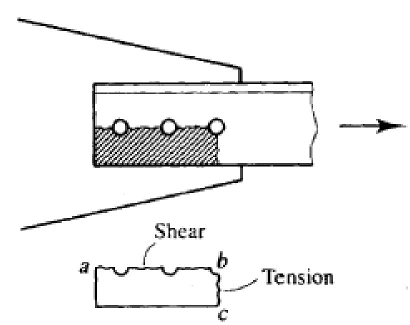
Staggered Fasteners

All possible failure patterns should be considered.



Block Shear

- Block shear is an important consideration in the design of steel connections. Consider the figure below that shows the connection of a single-angle tension member. The block is shown shaded.



Block Shear

- The nominal strength in tension is $F_u A_{nt}$ for fracture and $F_y A_{gt}$ for yielding where the second subscript t denotes area on the tension surface (bc in the figure above).
- The yield and ultimate stresses in shear are taken as 60% of the values in tension. The AISC Steel Manual considers two failure modes:
  - Shear yield - tension fracture vs Shear fracture - tension yield

$$T_n = 0.6F_y A_{gv} + U_{bs} F_u A_{nt} \leq T_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \qquad (J4\text{-}5)$$

- Because the limit state is fracture, the equation with the larger of the two fracture values controls where $\varphi_t = 0.75$.

Design of Tension Members

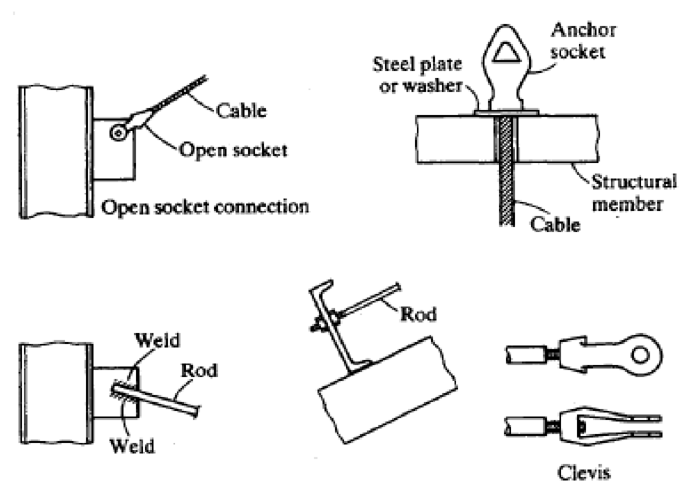
- The design of a tension member involves selecting a member from the AISC Steel Manual with adequate
  - Gross area
  - Net area
  - Slenderness (L/r≤300 to prevent vibration, etc; does not apply to cables.)
- If the member has a bolted connection, the choice of cross section must account for the area lost to the bolt holes.
- Because the section size is not known in advance, the default values of U are generally used for preliminary design.

Design of Tension Members

- Detailing of connections is a critical part of structural steel design. Connections to angles are generally problematic if there are two lines of bolts.
- Consider the Gages for Angle figure shown earlier that provides some guidance on sizing angles and bolts.
  - Gage distance g_1 applies when there is one line of bolts
  - Gage distances g_2 and g_3 apply when there are two lines

Design of Tension Members

Threaded Rod



Design of Tension Members

Threaded Rod -

- Tension on the effective net area

  $T_n = A_s F_u = 0.75 A_b F_u$

  where A_s is the stress area (threaded portion), A_b is the nominal (unthreaded area), and 0.75 is a lower bound (conservative) factor relating A_s and A_b. See Section J3.6 of the AISC Steel Manual Specification for details.

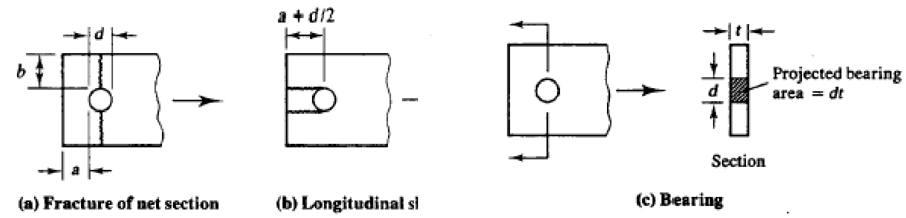
- The design strength of a threaded rod is calculated as

  $\varphi T_n = 0.75\ T_n$

Design of Tension Members

Pinned Connections

- Pinned connections transmit no moment (ideally) and often utilize components machined to tight tolerances (plus, minus 0.001”).
- The figure shows failure modes for pin-connected members and each failure mode must be checked for design. Specifically, the following limit states must be checked.



Design of Tension Members

The following limit states must be checked.

- Tension on the effective net area

  $\varphi T_n = 0.75(2\,t\,b_{eff} F_u)$ where $b_{eff} = 2t + 0.63 \leq b$ (D5-1)

- Shear on the effective area

  $\varphi T_n = 0.75(0.6 A_{sf} F_u) = 0.75\{0.6[2t + d/2)]\ F_u\}$ (D5-2)

- Bearing on projected area

  $\varphi T_n = 0.75(1.8\ A_{pb} F_y) = 0.75[1.8\ (d\ t\ )\ F_y\ ]$ (J8-1)

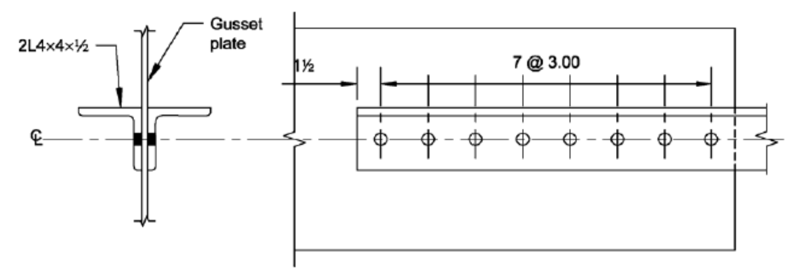
  where $1.8\ A_{pb} F_y$ is based on a deformation limit state under service loads producing stresses of 90% of yield

- Tension on the gross section

  $\varphi T_n = 0.9(A_g F_y)$ (D1-1)

Design Example of W-Shape Flexural Members

A 2L4×4×½ (⅜-in. separation), ASTM A36, has one line of (8) ¾-in. diameter bolts in standard holes and is 25 ft in length. The double angle is carrying a dead load of 40 kips and a live load of 120 kips in tension. Verify the strength by both LRFD and ASD.



Solution:

Material Properties:

2L4×4×½	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi	Manual Table 2-3

Geometric Properties:

For a single L4×4×½	$A_g = 3.75$ in.^{2}	$r_x = 1.21$ in.	Manual Table 1-7
	$r_y = 1.83$ in.	$\bar{x} = 1.18$ in.	

Design Example of Tension Members

- Calculate the required tensile strength
  $P_u = 1.2(40 \text{ kips}) + 1.6(120 \text{ kips}) = 240 \text{ kips}$
- Calculate the allowable tensile yield strength
  $P_u = F_y A_g = (36ksi)(2)(3.75in2) = 270 \text{ kips}$
  $\varphi P_u = 0.9(270) = 243 \text{ kips}$
- Calculate the available tensile rputure strength
  Calculate U: $U = 1 - \bar{x}/l = 1 - (1.18 \text{ in.}/21.0 \text{ in.}) = 0.944$
  Calculate A_n : $A_n = A_g - 2(d_b+1/16 \text{ in.})t = 2(3.75in^2) - 2(13/16 \text{ in.} + 1/16.) = 6.63 \text{ in}^2$
  Calculate A_e : $A_e = A_n U = 6.63 \text{ in}^2 (0.944) = 6.26 \text{ in}^2$
- Calculate the allowable tensile rupture strength
  $P_u = F_u A_e = (58ksi)(6.26in^2) = 363 \text{ kips}$
  $\varphi P_u = 0.75(363) = 272 \text{ kips}$
- The available strength is governed by the tensile yield limit state
  243 kips > 240 kips ***o.k.***

6300. Design -

6310. Structural Steel Members and Components

Objective and Scope Met

- Module 1: Tension
  - Introduction
  - Design strength
  - Net area
  - Staggered fasteners
  - Block shear
  - Design of tension members
  - Threaded rods, pin-connected members

6310. Structural Steel Members and Components – Module 2: Flexure and Shear

This section of the module covers:

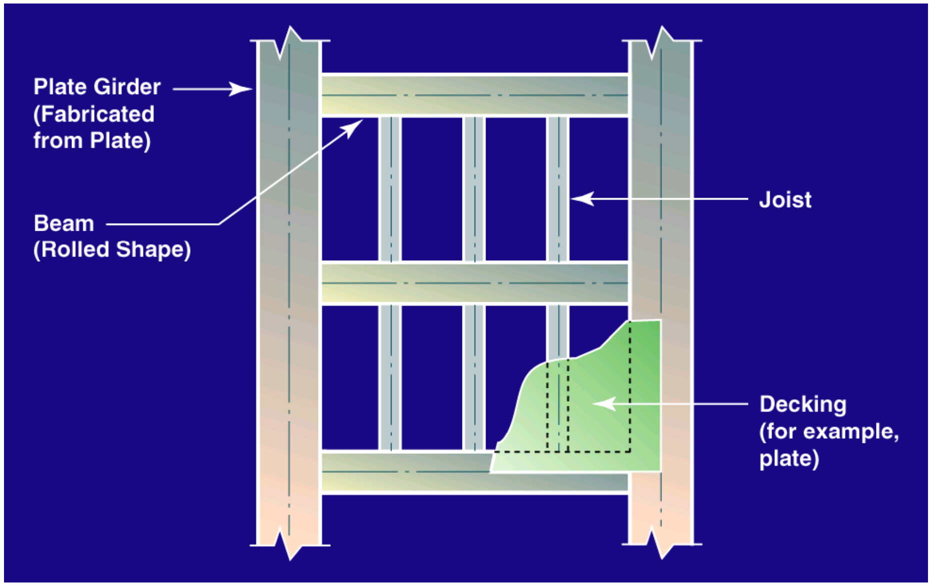
- Introduction
- Analysis
- Stability
  - Lateral Torsional Buckling (LTB)
  - Flange Local Buckling (FLB)
  - Web Local Buckling (WLB)
- Serviceability
- Shear strength
- Biaxial bending

Introduction to Flexure

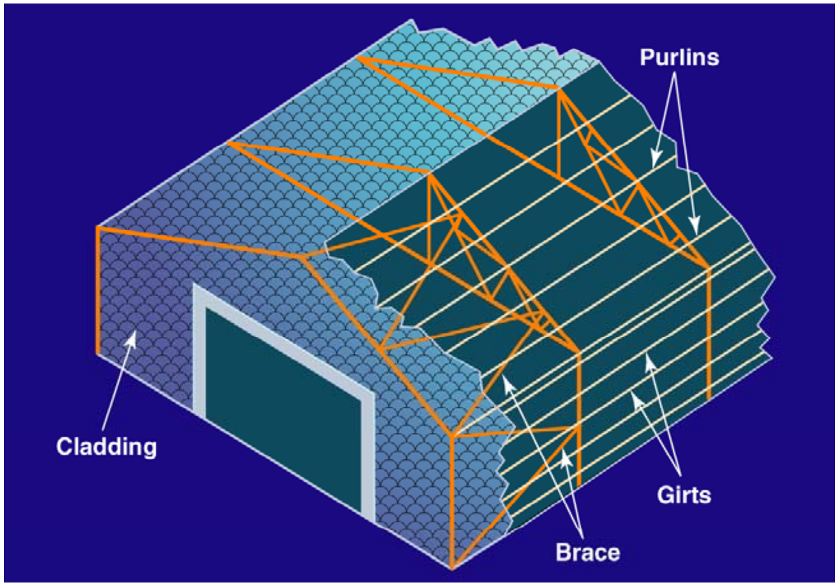
Components Subject to Lateral Loading

- Beams
- Girders
- Purlins
- Girts
- Joists
- Cladding

Example of a Typical Floor Plan



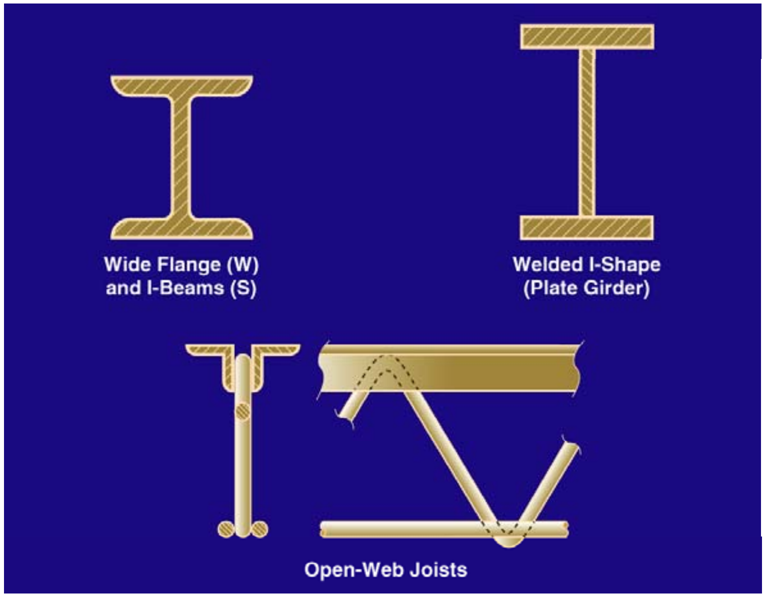
Example of a Typical Steel Structure



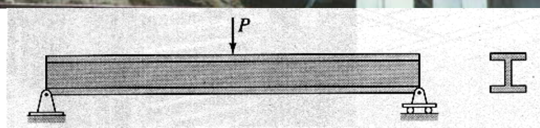
Introduction to Flexure

- Flexural members/beams are defined as members acted upon primarily by transverse loading, often gravity dead and live load effects. Thus, flexural members in a structure may also be referred to as:
  - Girders – usually the most important beams, which are frequently at wide spacing.
  - Joists – usually less important beams which are closely spaced, frequently with truss-type webs.
  - Purlins – roof beams spanning between trusses.
  - Stringers – longitudinal bridge beams spanning between floor beams.
  - Girts – horizontal wall beams serving principally to resist bending due to wind on the side of an industrial building, frequently supporting corrugated siding.
  - Lintels – members supporting a wall over window or door openings

Typical Beam Members



Types of Beams



Selecting Steel Beams and Girders

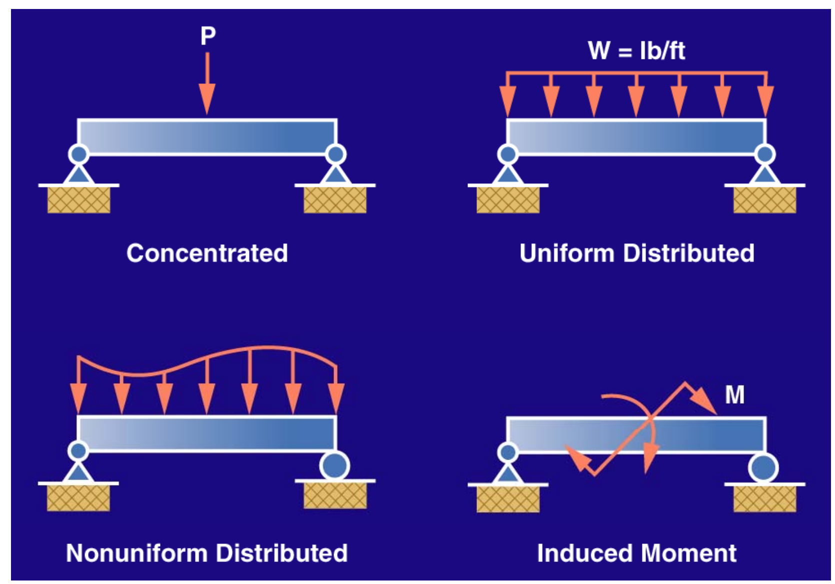
- Analysis and formulas for beams
- Types of flexural section and allowable stresses
- Compression flange considerations
- AISC rolled section selection tables
- Special considerations

Analysis and Formulas for Beams

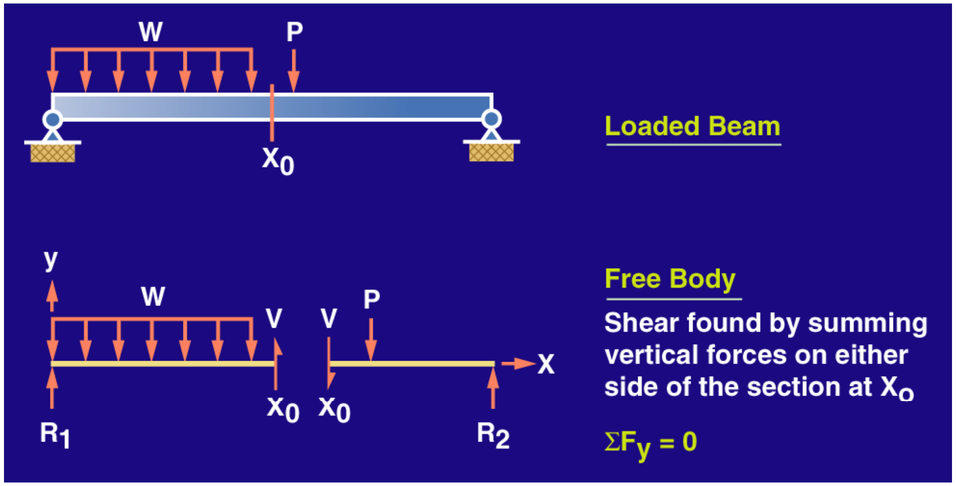
The following topics will be discussed:

» Load
» Shear
» Bending moment
» Stress
» Deflection

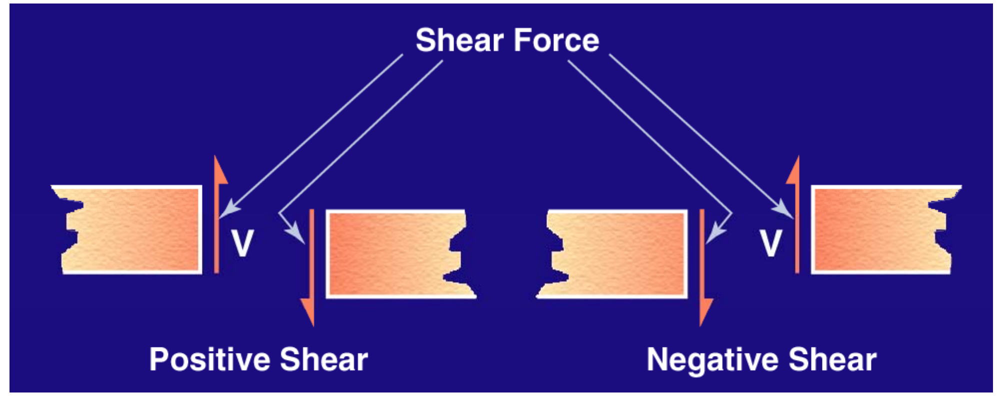
Four Basic Types of Loads



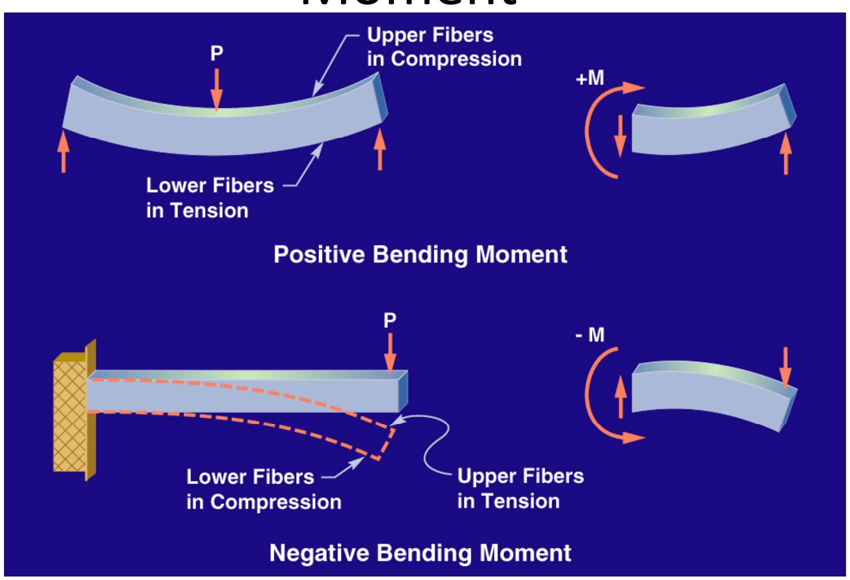
Vertical Shear Force



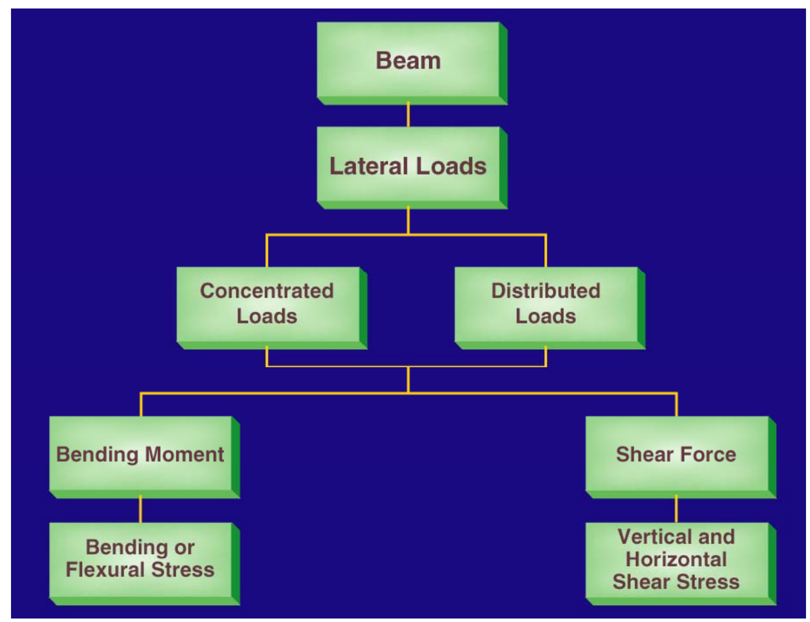
Positive and Negative Shear



Positive and Negative Bending Moment



Steps for Determining Stress



Formulas For Calculating Normal Bending Stress

$$\sigma = \frac{My}{I} \qquad \textbf{(Eqn. 9)}$$

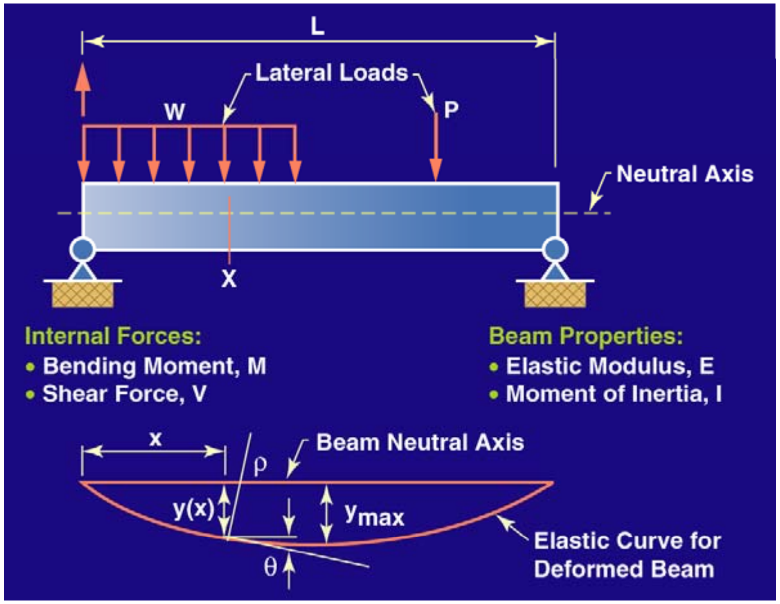
where: σ **= Bending stress**

M = Bending moment

y = Distance from neutral axis to fiber under consideration

I = Moment of inertia

Deflection



Stability

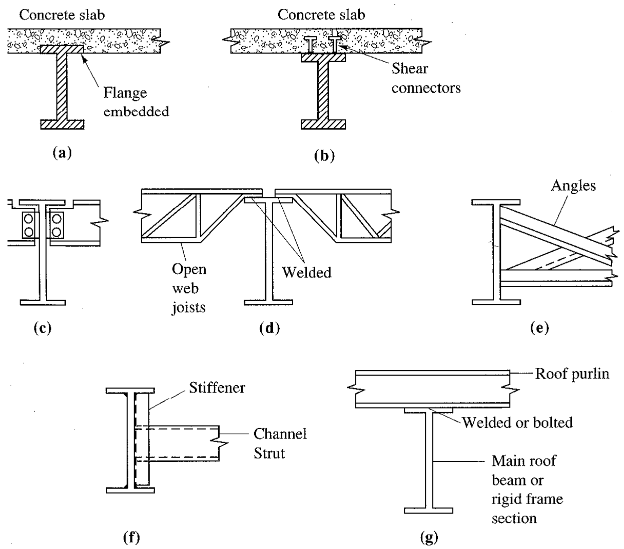
- The laterally supported beams assume that the beam is stable up to the fully plastic condition, that is, the nominal strength is equal to the plastic strength, or $M_n = M_p$
- If stability is not guaranteed, the nominal strength will be less than the plastic strength due to
  - Lateral-torsional buckling (LTB)
  - Flange and web local buckling (FLB & WLB)
- When a beam bends, one half (of a doubly symmetric beam) is in compression and, analogous to a column, will buckle.

Stability

- Unlike a column, the compression region is restrained by a tension region (the other half of the beam) and the outward deflection of the compression region (flexural buckling) is accompanied by twisting (torsion). This form of instability is known as lateral- torsional buckling (LTB)
- LTB can be prevented by lateral bracing of the compression flange. The moment strength of the beam is thus controlled by the spacing of these lateral supports, which is termed the unbraced length.

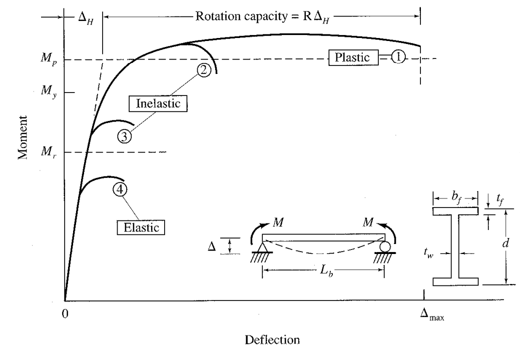
Stability

- Flange and web local buckling (FLB and WLB, respectively) must be avoided if a beam is to develop its calculated plastic moment.



Stability

- Four categories of behavior are shown in the figure:
  - Plastic moment strength M_p along with large deformation.
  - Inelastic behavior where plastic moment strength M_p is achieved but little rotation capacity is exhibited.
  - Inelastic behavior where the moment strength M_r, the moment above which residual stresses cause inelastic behavior to begin, is reached or exceeded.
  - Elastic behavior where moment strength M_{cr} is controlled by elastic buckling.



Types of Flexural Sections

Flexural sections are classified and described as:

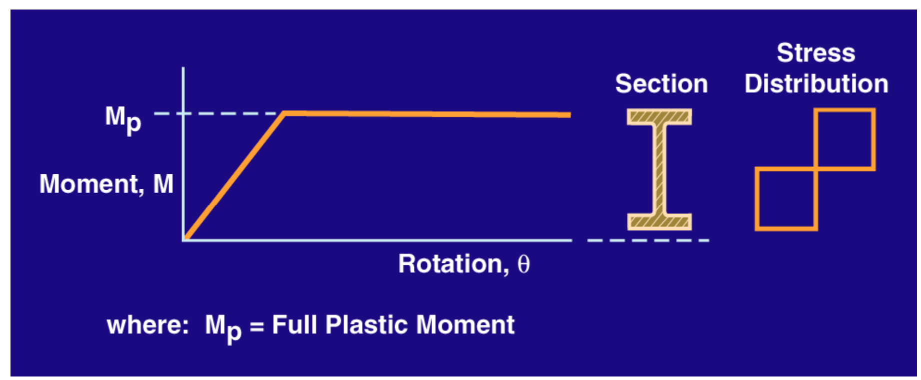
» Plastic

» Compact

» Noncompact

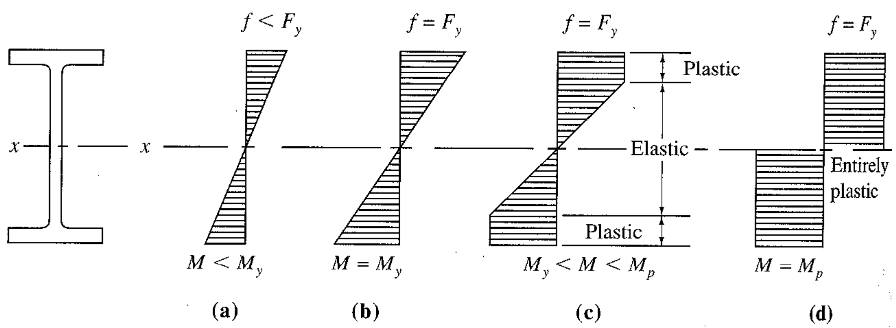
» Slender

Plastic Section



Laterally Supported Beams

- The stress distribution on a typical wide-flange shape subjected to increasing bending moment is shown below



Laterally Supported Beams

- In the service load range the section is elastic as in (a)
- When the yield stress is reached at the extreme fiber (b), the yield moment M_y is

  $M_n = M_y = S_x F_y$ (7.3.1)
- When the condition (d) is reached, every fiber has a strain equal to or greater than $\varepsilon_y = F_y/E_s$, the plastic moment M_p is

$$M_P = F_y \int_A y\,dA = F_y Z \qquad (7.3.2)$$

  Where Z is called the plastic modulus

Laterally Supported Beams

- Note that ratio, shape factor ξ, M_p/M_y is a property of the cross-sectional shape and is independent of the material properties.

  $\xi = M_p/M_y = Z/S$
- Values of S and Z (about both x and y axes) are presented in the Steel Manual Specification for all rolled shapes.
- For W-shapes, the ratio of Z to S is in the range of 1.10 to 1.15

Laterally Supported Beams

- The AISC strength requirement for beams:

  $\phi_b M_n \geq M_u$
  - Compact sections: $M_n = M_p = Z\,F_y$
  - Noncompact sections: $M_n = M_r = (F_y - F_r)\,S_x = 0.7F_y S_x$
  - Partially compact sections

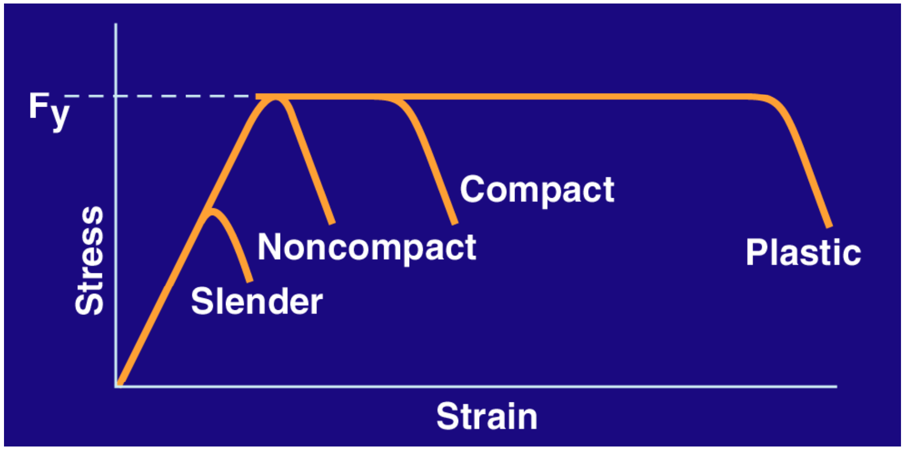
$$M_n = M_P - (M_p - M_r)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \leq M_P$$

  where $\lambda = b_f/2t_f$ for I-shaped member flanges

  $= h/t_w$ for beam web

  λ_r, λ_p from AISC Table B4.1
  - Slender sections: When the width/thickness ratio λ exceed the limits λ_r of AISC-B4.1

Stress vs. Strain Curves for Different Classes of Sections



Introduction of Beam Buckling

A beam can fail by reaching the plastic moment and becoming fully plastic (see last section) or fail prematurely by:

1. LTB, either elastically or inelastically
2. FLB, either elastically or inelastically
3. WLB, either elastically or inelastically

If the maximum bending stress is less than the proportional limit when buckling occurs, the failure is elastic. Else it is inelastic.

For bending $\varphi_b M_n (\varphi_b = 0.9)$

Design of Members for Flexure (about Major Axis)

TABLE User Note F1.1
Selection Table for the Application of Chapter F Sections

Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB
F3		NC, S	C	LTB, FLB
F4		C, NC, S	C, NC	Y, LTB, FLB, TFY
F5		C, NC, S	S	Y, LTB, FLB, TFY

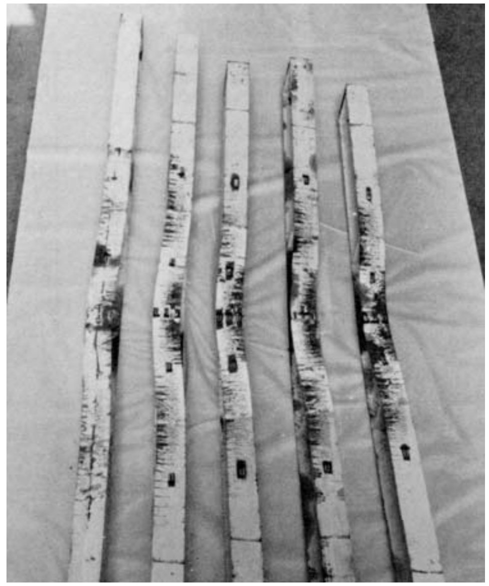
Lateral Torsional Buckling (LTB)

- Compact Members (AISC F2)
- Failure Mode
- Plastic LTB (Yielding)
- Inelastic LTB
- Elastic LTB
- Moment Gradient Factor C_b

Lateral Torsional Buckling (LTB)

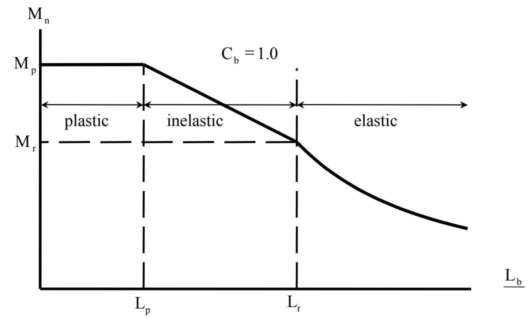
- Failure Mode

  A beam can buckle in a lateral-torsional mode when the bending moment exceeds the critical moment.



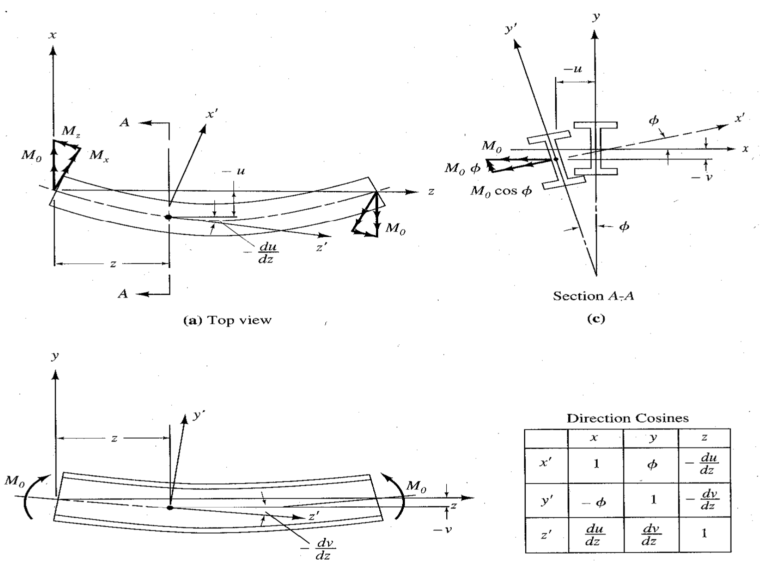
Lateral Torsional Buckling (LTB)

- Nominal Flexural Strength M_n
  - plastic when $L_b \le L_p$ and $M_n = M_p$
  - inelastic when $L_p < L_b \le L_r$ and $M_p > M_n \ge M_r$
  - elastic when $L_b > L_r$ and $M_n < M_r$



Lateral Torsional Buckling (LTB)

I-Beam in a Buckled Position



(a) Top view

Section A-A
(c)

(b) Side view

Direction Cosines

	x	y	z
x'	1	ϕ	$-\frac{du}{dz}$
y'	$-\phi$	1	$-\frac{dv}{dz}$
z'	$\frac{du}{dz}$	$\frac{dv}{dz}$	1

Lateral Torsional Buckling (LTB)

- Plastic LTB (Yielding)
  - Flexural Strength $M_n = M_p = F_y Z$ (AISC F2-1)

  where Z= plastic section modulus & F_y= section yield stress
  - Limits
    - Lateral bracing limit $L_b < L_p = 1.76 r_y \sqrt{\dfrac{E}{F_y}}$ (AISC F2-5)
    - Flange and Web width/thickness limit (AISC Table B4.1)

Lateral Torsional Buckling (LTB)

- Inelastic LTB $L_p < L_b \le L_r$
  - Flexure Strength (straight line interpolation)

$$M_n = C_b\left[M_p - \left(M_p - M_r\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \qquad (9.6.4)$$

or

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_y S_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \qquad \text{(AISC F2-2)}$$

Lateral Torsional Buckling (LTB)

- Elastic LTB $L_b > L_r$
  - Flexure Strength

$$M_n = F_{cr} S_x \le M_p \qquad \text{(AISC F2-3)}$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \qquad \text{(AISC F2-4)}$$

(The square root term may be conservatively taken equal to 1.0)
(c in AISC F2-8a,b for doubly symmetric I-shape, and channel, respectively)

  - Limit

$$L_r = 1.95 r_{ts}\frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_x h_o}}\sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y}{E}\frac{S_x h_o}{Jc}\right)^2}} \qquad \text{(AISC F2-6)}$$

$$r_{ts}^{\ 2} = \frac{\sqrt{I_y C_w}}{S_x} \qquad \text{(AISC F2-7)}$$

Lateral Torsional Buckling (LTB)

- Moment Gradient Factor C_b
  - The moment gradient factor C_b accounts for the variation of moment along the beam length between bracing points. Its value is highest, C_b=1, when the moment diagram is uniform between adjacent bracing points.
  - When the moment diagram is not uniform

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} \qquad \text{(AISC F1-1)}$$

where

M_{max}= absolute value of maximum moment in unbraced length

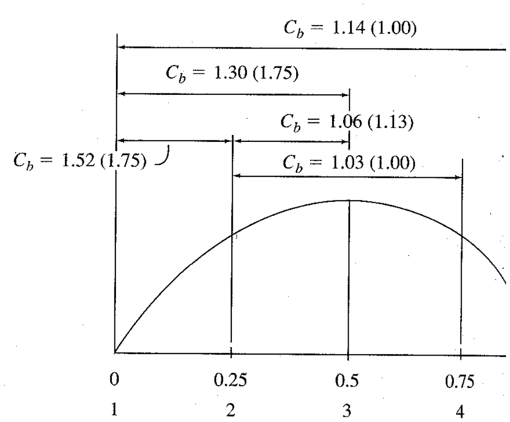
M_A, M_B, M_C= absolute moment values at one-quarter, one-half, and three-quarter points of unbraced length

C_b for a Simple Span Bridge

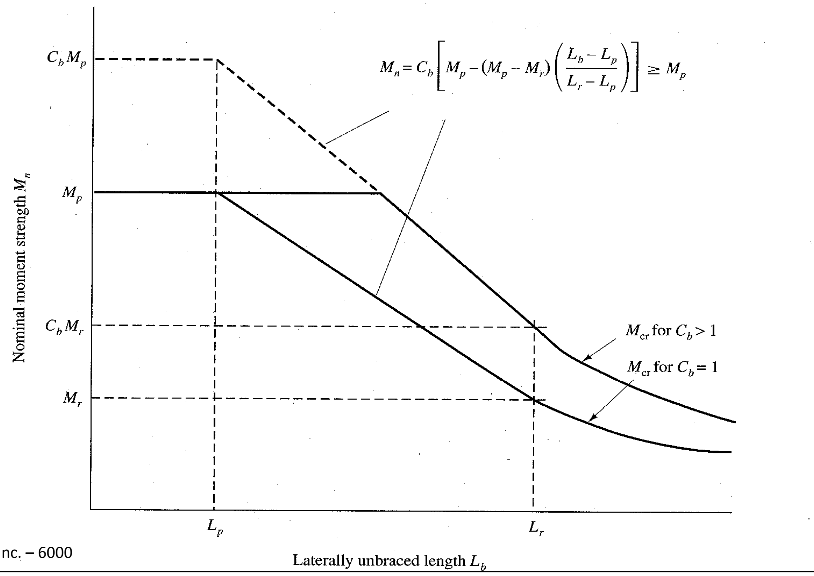
C_b FOR PARABOLIC SEGMENTS USING LRFD-F1.2a, FORMULA (C-F1-3), EQ. 9.6.11*

Case 1	Laterally braced at ends; points 1 and 5 only; $M_{\max}$ at 3	$C_b = 1.14$
Case 2	Laterally braced at ends and midspan; points 1,3, and 5 only; $M_{\max}$ at 3	$C_b = 1.30$
Case 3	Laterally braced at end and 1st quarter point; bracing at points 1 and 2; $M_{\max}$ at 2	$C_b = 1.52$
Case 4	Laterally braced at 1st and 2nd quarter points; bracing at points 2 and 3; $M_{\max}$ at 3	$C_b = 1.06$
Case 5	Laterally braced at 1st and 3rd quarter points; bracing at points 2 and 4; $M_{\max}$ at 3	$C_b = 1.03$

* Values from 1986 LRFD, Eq. 9.6.12 shown in parenthesis.



Nominal Moment Strength M_n as affected by C_b



Flange Local Buckling (FLB)

- Compact Web and Noncompact/Slender Flanges (AISC F3)
- Failure Mode
- Noncompact Flange
- Slender Flange
- Nominal Flexural strength, M_n = Min (F2, F3)

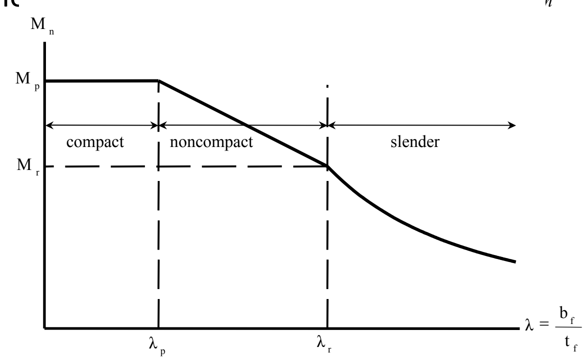
Flange Local Buckling (FLB)

- Failure Mode

  The compression flange of a beam can buckle locally when the bending stress in the flange exceeds the critical stress.

Flange Local Buckling (FLB)

- Nominal Flexural Strength M_n
  - plastic when $b/2t_f \leq \lambda_p$ and $M_n = M_p$
  - inelastic when $\lambda_p \leq b/2t_f \leq \lambda_r$ and $M_p > M_n \geq M_r$
  - elastic when $b/2t_f > \lambda_r$ and $M_n < M_r$



Flange Local Buckling (FLB)

- Noncompact Flange (straight line interpolation)
  – Flexure Strength

$$M_n = M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \text{(AISC F3-1)}$$

Flange Local Buckling (FLB)

- Slender Flange
  – Flexure Strength

$$M_n = \frac{0.9Ek_cS_x}{\lambda^2} \quad \text{(AISC F3-2)}$$

$$k_c = \frac{4}{\sqrt{h/t_w}}$$

(kc shall not be less than 0.35 and not greater than 0.76)

  – Limit (AISC Table B4.1)

Web Local Buckling (WLB)

- Compact or Noncompact Webs (AISC F4)
- Failure Mode
- Compact Web (Yielding)
- Noncompact Web
- Slender Web
- Nominal Flexural Strength, M_n=min (compression flange yielding, LTB, compression FLB, tension flange yielding)

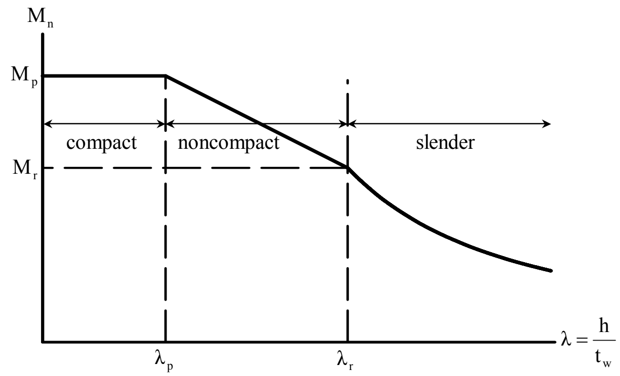
Web Local Buckling (WLB)

- Failure Mode

  The web of a beam can also buckle locally when the bending stress in the web exceeds the critical stress.

Web Local Buckling (WLB)

- Nominal Flexural Strength M_n
  - plastic when $\lambda \le \lambda_p$ and $M_n = M_p$
  - inelastic when $\lambda_p < \lambda \le \lambda_r$ and $M_p > M_n \ge M_r$
  - elastic when $\lambda > \lambda_r$ and $M_n < M_r$



Web Local Buckling (WLB)

- Compression Flange Yielding
  - Flexural Strength

$$M_n = R_{pc} M_{yc} = R_{pc} F_y S_{xc} \qquad \text{(AISC F4-1)}$$

where R_{pc}= web plastification factor (AISC F4-9a, b) & F_y= section yield stress

  - Limits (AISC Tables B4.1)

$$L_b < L_p = 1.1 r_t \sqrt{\frac{E}{F_y}}$$

Web Local Buckling (WLB)

- LTB (Inelastic) $L_p < L_b \le L_r$
  - Flexure Strength

$$M_n = C_b \left[R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{xc} \right) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \le M_p \qquad \text{(AISC F4-12)}$$

where F_L= a stress determined by AISC F4-6a, b

Web Local Buckling (WLB)

- LTB (Elastic) $L_b > L_r$
  - Flexure Strength

$$M_n = F_{cr} S_{xc} \le R_{pc} M_{yc} \qquad \text{(AISC F4-3)}$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t} \right)^2} \sqrt{1 + 0.078 \frac{J}{S_x h_o} \left(\frac{L_b}{r_t} \right)^2} \qquad \text{(AISC F4-5)}$$

  - Limit (AISC Table B4.1)

$$L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{F_L}{E} \frac{S_x h_o}{J} \right)^2}} \qquad \text{(AISC F4-8)}$$

Shear Strength

- Failure Mode
- Shear-Buckling Coefficient
- Elastic Shear Strength
- Inelastic Shear Strength
- Plastic Shear Strength

For shear $\varphi_v V_n$(φ_v = 0.9 except certain rolled I-beam $h/t_w \leq 2.24\sqrt{E/F_y}$, $\varphi_v = 1.0$)

$V_n = 0.6F_yA_wC_v$ (AISC G2-1)

Shear Strength

- Failure Mode

  The web of a beam or plate girder buckles when the web shear stress exceeds the critical stress.

Shear on Rolled Beams

- General Form $v = VQ/(It)$ and average form is

  $f_v = V/A_w = V/(dt_w)$
- AISC-F2

  $\phi_v V_n \geq V_u$

  where

  $\phi_v = 1.0$

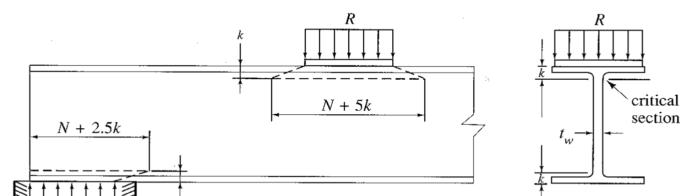
  $V_n = 0.6F_{yw}A_w$ for beams without transverse stiffeners and $h/t_w \leq 2.24/\sqrt{E/F_y}$

Concentrated Loads

- AISC-J10.2 $\phi R_n \geq R_u$
  - Local web yielding (use R_1 & R_2 in AISC Table 9-4)
    1. Interior loads

       $R_n = (5k + N)F_{yw}t_w$
    2. End reactions

       $R_n = (2.5k + N)F_{yw}t_w$ (7.8.3)



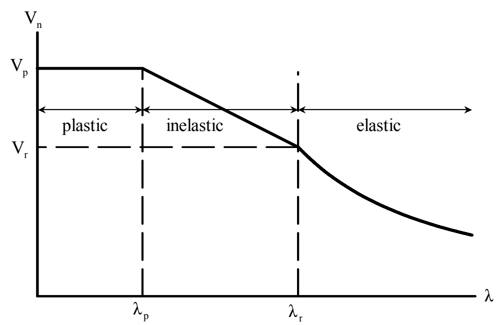
N = bearing length

k = distance from outer face of flange to web toe of fillet (property given AISC Manual with dimensions of rolled sections)

R = concentrated load to be transmitted to girder

Shear Strength

- Nominal Shear Strength V_n $(\varphi_v = 0.9)$
  - plastic when $\lambda \le \lambda_p$ and $\tau = \tau_y$
  - inelastic when $\lambda \le \lambda_r$ and $\tau = 0.8\tau_y$
  - elastic when $\lambda > \lambda_r$ and $\tau = \tau_{cr}$



Shear Strength

- AISC G2 Nominal Shear Strength V_n

(a) For $\dfrac{h}{t_w} \le 1.10\sqrt{\dfrac{k_c E}{F_{yw}}}$ $C_v = 1.0$ (AISC G2-3)

(a) For $1.10\sqrt{\dfrac{k_c E}{F_y}} \le \dfrac{h}{t_w} \le 1.37\sqrt{\dfrac{k_c E}{F_y}}$ $C_v = \left[\dfrac{1.10\sqrt{k_v E / F_y}}{h/t_w}\right]$ (AISC G2-4)

(a) For $1.37\sqrt{\dfrac{k_c E}{F_{yw}}} \le \dfrac{h}{t_w}$ $C_v = \left[\dfrac{1.51 E k_v}{\left(h/t_w\right)^2 F_y}\right]$ (AISC G2-5)

Special Considerations for Designing Flexural Members

- Deflection
- Vibration
- Ponding

Serviceability of Beam

- Deflection
  - AISC – Section L3: Deformations in structural members and structural system due to service loads shall not impair the serviceability of the structure
  - ASD - $\Delta_{max} = 5wL^4/(384EI)$

    As a guide in ASD –Commentary L3.1

    - L/240 (roof); L/300 (architectural); L/200 (movable components)

    Past guides (still useful) listed in Salmon & Johnson

    - Floor beams and girders $L/d \le 800/F_y$, ksi

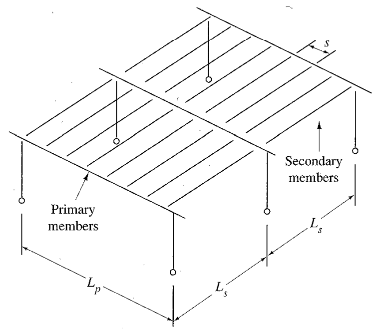
      to shock or vibratory loads, large open area $L/d \le 20$

    - Roof purlins, except flat roofs, $L/d \le 1000/F_y$

Serviceability of Beam

- Ponding (AISC Appendix 2, Sec. 2.1)
  $C_p + 0.9C_s \leq 0.25$

  $I_d \geq 25(s^4)10^{-6}$
  where
  - $C_p = 32L_sL_p^4/(10^7I_p)$
  - $C_s = 32SL_s^4/(10^7I_s)$
  - L_p = Column spacing in direction of girder
  - L_s = Column spacing perpendicular to direction of girder
  - I_p = moment of inertia of primary members
  - I_s = moment of inertia of secondary members
  - I_d = moment of inertia of the steel deck



Purlins and Girts

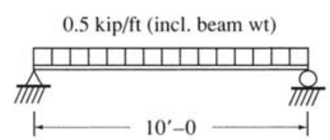
Purlins and girts have the same design procedures as beams but are lighter due to reduced loading requirements. They are used in building walls and roofs. The AISC Is a source of design data

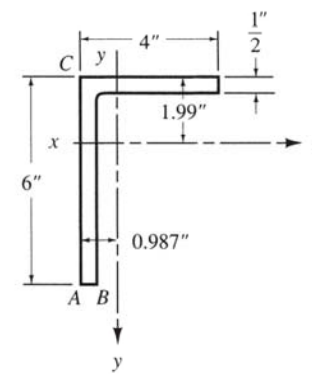
Cladding

Sources of design data for cladding are:

- American Iron and Steel Institute, Cold-Formed Steel Design Manual
- Manufacturers' handbooks & product manuals, for example, Whirlwind Building Systems

General Flexural Theory





$$\sigma \leq \frac{M_xI_y - M_yI_{xy}}{I_xI_y - I_{xy}^2}y + \frac{M_yI_x - M_xI_{xy}}{I_xI_y - I_{xy}^2}x$$

(a) Angle free to bend in any direction

(b) Angle restrained to bend in the vertical plane

Biaxial Bending of Symmetric Sections

- AISC-H2

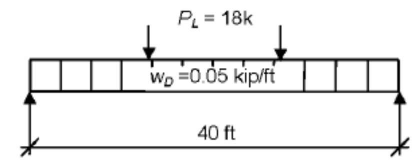
$$\frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1$$

$$S_x \leq \frac{M_{ux}}{\phi_b F_y} + \frac{M_{uy}}{\phi_b F_y}\left(\frac{S_x}{S_y}\right)$$

Design Example of Tension Members

Given:

Select an ASTM A992 W-shape beam with a simple span of 40 feet. The nominal loads are a uniform dead load of 0.05 kip/ft and two equal 18 kip concentrated live loads acting at the third points of the beam. The beam is continuously braced. Also calculate the deflection.



Beam Loading & Bracing Diagram
(Continuous bracing)

Note: A beam with noncompact flanges will be selected to demonstrate that the tabulated values of the *Steel Construction Manual* account for flange compactness.

Solution:

Material Properties:

ASTM A992 $F_y = 50$ ksi $F_u = 65$ ksi

Design Example of Tension Members

By AISC Steel Manual

- Calculate the required flexural strength at midspan

  $w_u = 1.2(0.05 \text{ kip/ft}) = 0.06 \text{ kip/ft}$; $P_u = 1.6(18 \text{ kips}) = 28.8 \text{ kips}$

  $M_u = (0.06 \text{ kip/ft})(40 \text{ ft})^2/8 + (28.8 \text{ kips})40 \text{ ft}/3 = 396 \text{ kip-ft}$

- By AISC Steel Manual: Select the lightest section with the required strength from the bold entries in Manual Table 3-2. Select W21x48 with noncompact compression flange at F_y=50 ksi (S_x = 93.0 in^3 & Z_x = 107 in^3 & $\lambda = b_f/2t_f$ = 9.47)

  $\varphi_b M_u = \varphi_b M_{px} = 398 \text{ kip-ft} > 396 \text{ kip-ft}$. ***o.k.***

Design Example of Tension Members

Verified by Calculation using the provisions of the Specification

- The limiting width-thickness ratios for the compression flange are:

  $\lambda_{pf} = 0.38 \sqrt{E/Fy} = 0.38 \sqrt{(29{,}000 \text{ ksi}/50 \text{ ksi})} = 9.15$

  $\lambda_{rf} = 1.00 \sqrt{E/Fy} = 1.00 \sqrt{(29{,}000 \text{ ksi}/50 \text{ ksi})} = 24.1$

  $\lambda_{rf} > \lambda > \lambda_{pf}$, therefore, the compression flange is noncompact

- Calculate the nominal flexural strength M_n

  $M_p = F_y Z_x = 50 \text{ ksi } (107 \text{ in.}^3) = 5350 \text{ kip-in. or } 446 \text{ kip-ft}$

  $$M_n = M_P - (M_p - M_r)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \leq M_P = 5350 - (5350 - 0.7(50)(93.0))\left(\frac{9.47 - 9.15}{24.1 - 9.15}\right)$$

  $M_n = 5310 \text{ kip-in. or } 442 \text{ kip-ft.}$

- Calculate the available flexural strength

  $\varphi_b M_p = 0.9(442 \text{ kip-ft.}) = 398 \text{ kip-ft} > 396 \text{ kip-ft}$. ***o.k.***

6300. Design -

6310. Structural Steel Members and Components

Objective and Scope Met

- Module 2: Flexure and Shear
  - Introduction
  - Analysis
  - Stability
    - Lateral Torsional Buckling (LTB)
    - Flange Local Buckling (FLB)
    - Web Local Buckling (WLB)
  - Serviceability
  - Shear strength
  - Biaxial bending

6310. Structural Steel Members and Components – Module 3: Compression

This section of the module covers:

- Introduction
- Design factors
- Load and member forces
- Stability and end-support considerations
- AISC-allowable stress and load tables
- Parameters and format of column design tables
- Design examples of columns

Compression

- Compression (Section NE and use of AISC Manual Part 4 - Column Design Table)

Definition of Columns

Columns:

- Are linear structural members loaded primarily along their longitudinal axis
- Have a uniform cross section (usually)
- Are oriented vertically in a structure
- Are often connected to beams and other Structural members

Introduction to Compression

- Axial Compression
  - Generally referred to as: "compression members" because the compression forces or stresses dominate their behavior.
  - In addition to the most common type of compression members (vertical elements in structures), compression members include:
    - Arch ribs
    - Rigid frame members inclined or otherwise
    - Compression elements in trusses
    - shells

Introduction to Compression

Introduction to Compression

- General
  - Columns include top chords of trusses and various bracing members.
  - In many cases, many members have compression in some of their parts. These include:
    - The compression flange
    - Built-up beam sections, and
    - Members that are subjected simultaneously to bending and compressive loads.

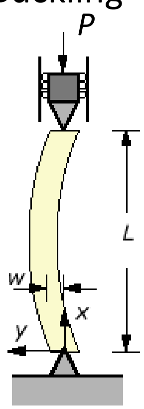
Introduction to Compression

- General
  - Mode of Failures for Columns
    1. **Flexural Buckling** (also called Euler buckling) is the primary type of buckling. Members are subject to flexure or bending when they become unstable.
    2. **Local Buckling:** This type occurs when some part or parts of the cross section of a column are so thin that they buckle locally in compression before the other modes of buckling can occur. The susceptibility of a column to local buckling is measured by the width-thickness ratio of the parts of the cross section

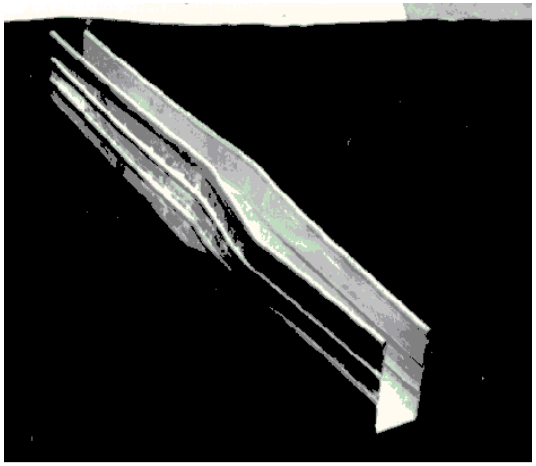
Introduction to Compression

- General

Euler Buckling

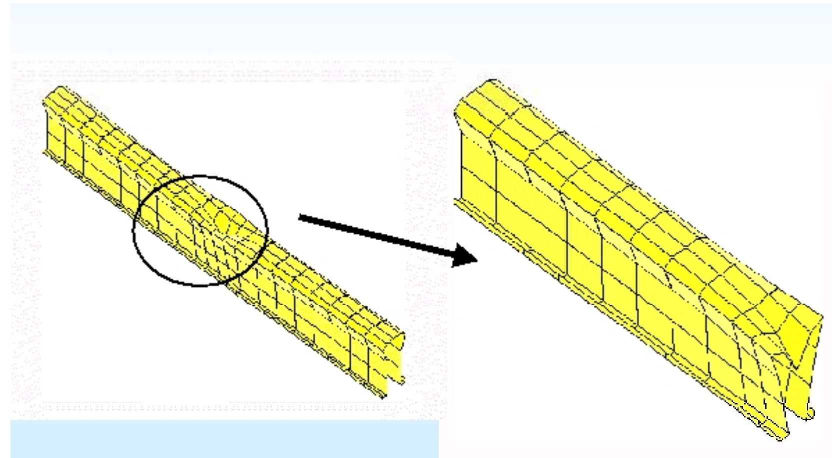


*Simply supported column subjected to axial load **P***



Introduction to Compression

- General
  - Local Buckling



Introduction to Compression

- General
  - Mode of Failures for Columns (cont'd)
    3. **<u>Torsional Buckling</u>** may occur in columns that have certain cross-sectional configurations. These columns fail by twisting (torsion) or by a combination of torsional and flexural buckling.

Introduction to Compression

- Why is a column more critical than a beam or a tension member?
  - A column is a more critical member in a structure than is a beam or tension members because minor imperfections in materials and dimensions mean a great deal.
  - This fact can be illustrated by a bridge truss that has some of its members damaged by a truck.

Introduction to Compression

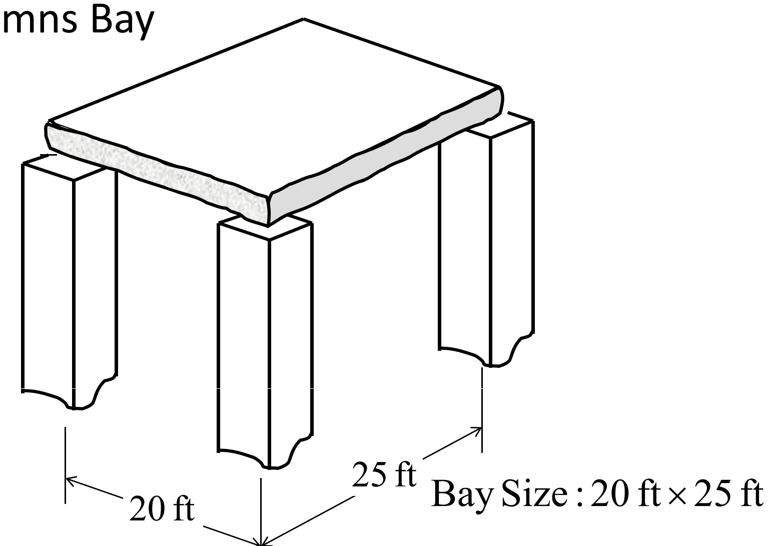
- Why is a column more critical than a beam or a tension member? (cont'd)
  – The bending of tension members probably will not be serious as the tensile loads will tend to straighten those members; but the bending of any compression members is a serious matter, as compressive loads will tend to magnify the bending in those members.

Introduction to Compression

- Columns Bay
  – The spacing of columns in plan establishes what is called a ***Bay***.
  – For example, if the columns are 20 ft on center in one direction and 25 ft in the other direction, the bay size is 20 ft × 25 ft.
  – Larger bay sizes increase the user's flexibility in space planning.

6310. Structural Steel Members and Components – Introduction to Compression

- Columns Bay



Design Factors

The two most important design factors in structural analysis of beams and columns are:

- Strength
- Stability

A third design factor for columns is:

- Serviceability

Design Factors

The parameters that can control or affect the behavior of a column are:

- Load magnitude, P
- Load eccentricity, e
- Area of cross section, A
- Radius of gyration r
- Effective length, $KL = L_e$
- End-support conditions
- Initial straightness
- Residual stress

Design Factors

- **Slenderness, $\frac{L}{r}$**
- **Material yield stress, σ_y, and ultimate stress, σ_u**
- **Material elastic modulus, E**

Column Slenderness

Based on the slenderness of a column, columns are classified as:

- Short
- Long
- Intermediate

Column Slenderness

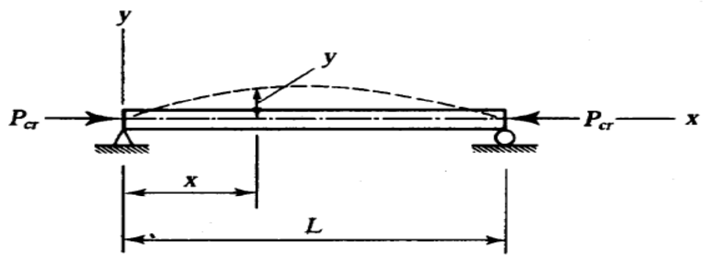
- Slenderness Ratio
  - The longer the column becomes for the same cross section, the greater becomes its tendency to buckle and the smaller becomes the load it will carry.
  - The tendency of a member to buckle is usually measured by its slenderness ratio, that is

$$\text{Slenderness Ratio} = \frac{L}{r} \qquad (1)$$

$$\text{where } r = \sqrt{\frac{I}{A}} = \text{radius of gyration}$$

Buckling Load

- If the axial load *P* is applied slowly, it will ultimately become large enough to cause the member to become unstable and assume the shape shown by the dashed line.
- The member has then buckled and the corresponding load is termed the critical buckling load (also termed the Euler buckling load after the mathematician Euler who formulated the relationship in 1759).



The Euler Formula

Critical Buckling Load and Stress

- Many columns lie between these extremes in which neither solution is applicable.
- These intermediate-length columns are analyzed by using empirical formulas to be described later.
- When calculating the critical buckling for columns, I (or r) should be obtained about the weak axis.

The Euler Formula

- Example 1

  A W10 × 22 is used as a 15-long pin-connected column. Using Euler expression (formula),

  a. Determine the column's critical or buckling load, assuming the steel has a proportional limit of 36 ksi.

  b. Repeat part (a) if the length of the column is changed to 8 ft.

The Euler Formula

- Example 1 (cont'd)

  Using a W10 × 22, the following properties can be obtained from the LRFD Manual:

  $A = 6.49$ in^2, $r_x = 4.27$ in, and $r_x = 1.33$ in

  Therefore, minimum $r = r_y = 1.33$ in.

a.
$$\frac{L}{r} = \frac{15 \times 12}{1.33} = 135.34$$

$$F_e = \frac{\pi^2 E}{(L/r)^2} = \frac{\pi^2 \left(29 \times 10^3\right)}{(135.34)^2} = 15.63\,\text{ksi} < 36\,\text{ksi}$$

OK column is in elastic range

The Euler Formula

- Example 1 (cont'd)
  b. Using an 8-ft W10 × 22:

$$\frac{L}{r} = \frac{8 \times 12}{1.33} = 72.18$$

$$F_e = \frac{\pi^2 E}{(L/r)^2} = \frac{\pi^2 (29 \times 10^3)}{(72.18)^2} = 54.94\ \text{ksi} > 36\ \text{ksi}$$

$\therefore$ column is in inelastic range and Euler equation is not applicable

Residual Stresses

- Residual stresses are stresses that remain in a member after it has been formed into a finished product.
- Causes:
  1. Uneven cooling that occurs after hot rolling of structural shapes.
  2. Cold bending or cambering during fabrication.
  3. Punching of holes during fabrication.
  4. Welding.

Residual Stresses

- Residual Stresses in Rolled Sections
  - In wide-flange or H-shaped sections, after hot rolling, the flanges, being the thicker parts, cool more slowly than the web region.
  - Furthermore, the flange tips having greater exposure to the air cool more rapidly than the region at the junction of the flange and the web.
  - Consequently, compressive residual stress exists at flange tips and mid-depth of the web, while tensile residual stress exists in the flange and the web at the regions where they join.

Residual Stresses

- Residual Stresses in Rolled Sections

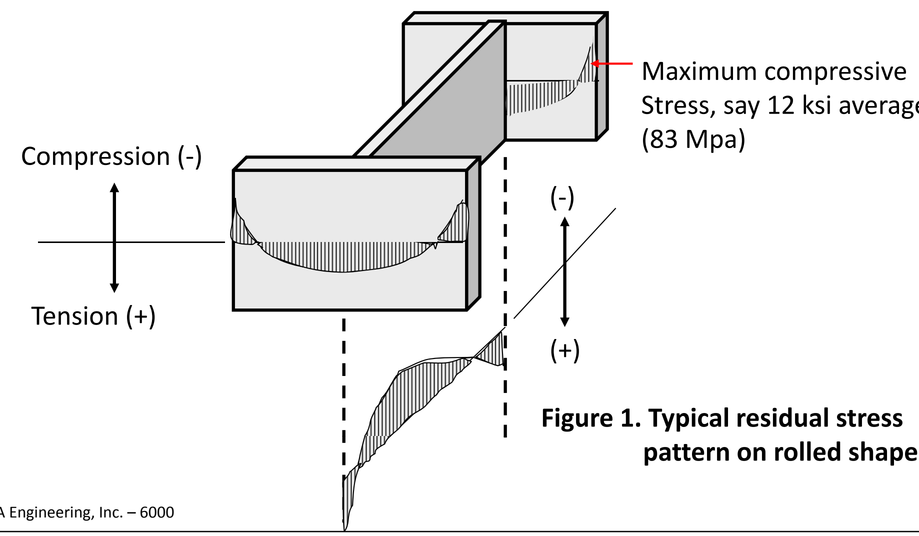


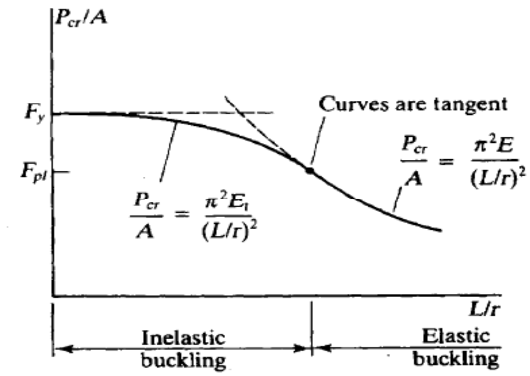
Figure 1. Typical residual stress pattern on rolled shapes

Material Imperfections

- Effect of Material Imperfections and Flaws
  - Slight imperfections in tension members and beams can be safely disregarded as they are of little consequences.
  - On the other hand, slight defects in columns may be of major significance.
  - A column that is slightly bent at the time it is put in place may have significant bending moment resulting from the load and the initial lateral deflection.

Buckling Stress vs Slenderness

- The critical buckling stress is often plotted as a function of slenderness as shown in the figure below. This curve is called a Column Strength Curve. From this figure it can be seen that the tangent modulus curve is tangent to the Euler curve at the point corresponding to the proportional limit.



Stability and End-Support Considerations

This section covers the following topics:

- Types of end supports
- Slenderness ratio, K factors, and effective lengths
- Sideway effect
- Moment magnification effects

Types of End Supports

Common member End conditions	Rotation fixed and translation fixed
	Rotation free and translation fixed
	Rotation fixed and translation free
	Rotation free and translation free

Slenderness Ratio

Slenderness ratio $= \frac{L}{r}$

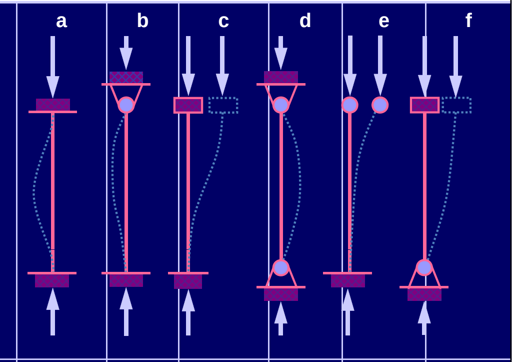
The effective length of a compression member is:

$$L_e = KL$$

The slenderness ratio then becomes:

$$\frac{L_e}{r} = \frac{KL}{r}$$

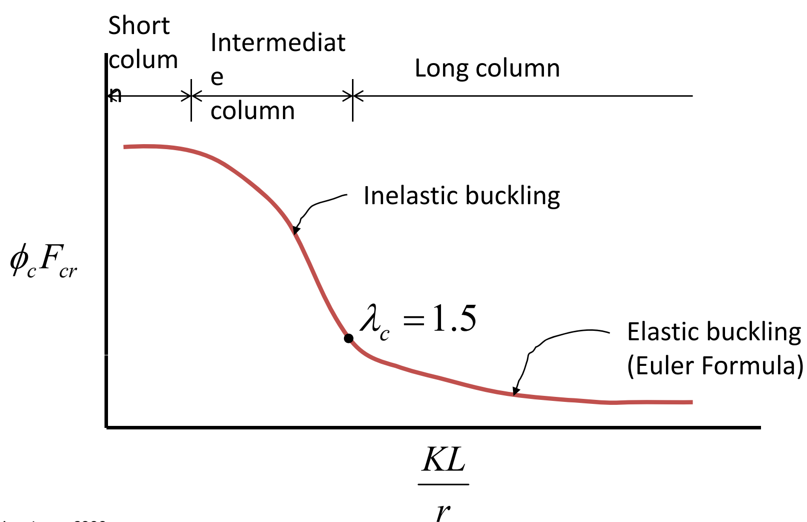
K Values for Support Conditions

	a	b	c	d	e	f
Dashed line show buckled shape of column						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when Ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0

End condition code:
- Rotation fixed and translation fixed
- Rotation free and translation fixed
- Rotation free and translation free
- Rotation free and translation free

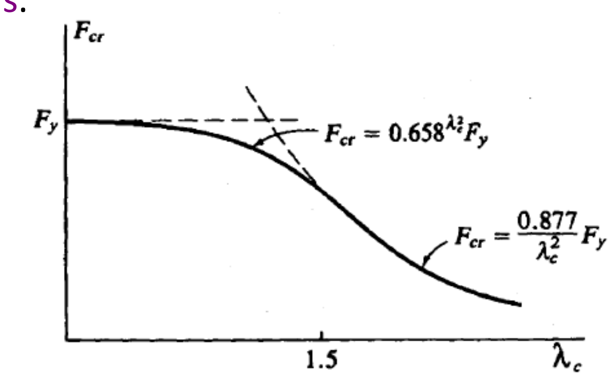
Column Formulas

Figure 1. LRFD Critical Buckling Stress



Column Design per AISC

- The above equations for the critical buckling stress are given in Section E.2 of the specification.
- The figure below illustrates the above equations and the transition point. AISC specifies a maximum slenderness ratio, *KL/r*, of 200 for compression members.



Column Design per AISC

Flange and web compactness

- For the strength associated with a buckling mode to develop, local buckling of elements of the cross section must be prevented. If local buckling (flange or web) occurs,
  - The cross-section is no longer fully effective.
  - Compressive strengths given by F_{cr} must be reduced
- Section B5 of the LRFD specification provides limiting values of width-thickness ratios (denoted λ_r) where shapes are classified as
  - Compact
  - Noncompact
  - Slender

Column Design per AISC

- AISC writes that if exceeds a threshold value λ_r, the shape is considered slender and the potential for local buckling must be addressed.
- Two types of elements must be considered
  - Unstiffened elements - Unsupported along one edge parallel to the direction of load

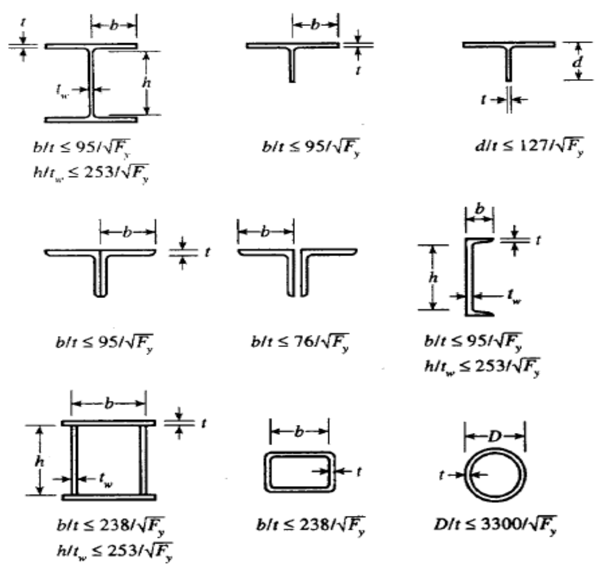
    (AISC LRFD Table B5.1, p 16.1-14)
  - Stiffened elements - Supported along both edges parallel to the load

    (AISC LRFD Table B5.1, p 16.1-15)

Column Design per AISC

The figure on the following page presents compression member limits (λ_r) for different cross-section shapes that have traditionally been used for design.

(AISC LRFD Fig. C-B5.1, p16.1-183)



Column Design per AISC

Tables for design of compression members -

- Tables 4.2 through 4.17 in Part 4 of the AISC LRFD specification present design strengths in axial compression for columns with specific yield strengths, for example, 50 ksi for W shapes. Data are provided for slenderness ratios of up to 200.
- Sample data are provided on the following page for some W14 shapes

Column Design per AISC

W14 samples

(AISC LRFD p 4-21)

Table 4-2 (cont.).
W-Shapes
Design Strength in Axial Compression, $\phi_c P_n$ kips

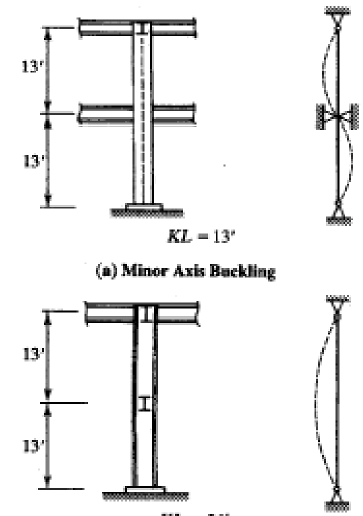
$F_y = 50$ ksi
$\phi_c P_n = 0.85 F_{cr} A_g$

Shape	W14×									
	311*	283*	257*	233*	211	193	176	159	145	132
0	3880	3540	3210	2910	2640	2410	2200	1980	1810	1650
11	3610	3290	2980	2700	2440	2230	2030	1830	1670	1510
12	3560	3240	2940	2660	2400	2200	2000	1810	1650	1480
13	3510	3200	2890	2620	2370	2170	1970	1780	1620	1450
14	3460	3140	2850	2570	2330	2130	1940	1740	1590	1430
15	3400	3090	2800	2530	2280	2090	1900	1710	1560	1390
16	3330	3030	2740	2480	2240	2050	1860	1680	1530	1360
17	3270	2970	2690	2430	2190	2010	1820	1640	1500	1330
18	3200	2910	2630	2380	2140	1960	1780	1600	1460	1300
19	3130	2850	2570	2320	2090	1910	1740	1570	1430	1260
20	3060	2780	2510	2270	2040	1870	1700	1530	1390	1220
22	2910	2640	2380	2150	1940	1770	1610	1440	1320	1150
24	2750	2500	2250	2030	1830	1670	1510	1360	1240	1070
26	2590	2350	2120	1910	1710	1560	1420	1270	1160	997
28	2430	2200	1980	1780	1600	1460	1320	1180	1080	920
30	2270	2050	1840	1660	1490	1350	1220	1100	996	844
32	2110	1900	1710	1530	1370	1250	1130	1010	919	769
34	1950	1760	1570	1410	1260	1150	1040	928	842	697
36	1790	1620	1440	1290	1160	1050	946	846	767	627
38	1640	1480	1320	1180	1050	965	859	767	694	563
40	1490	1340	1190	1070	951	883	775	692	626	508
42	1350	1220	1080	967	863	783	703	628	568	461
44	1230	1110	987	881	786	713	641	572	518	420
46	1130	1010	903	806	719	652	586	523	474	384
48	1040	932	829	741	660	599	538	481	435	353
50	956	858	764	682	609	552	496	443	401	325
Properties										
P_{wo}, kips	1010	861	735	621	529	454	396	333	287	263
P_{wi}, kips/in.	79.5	64.5	58.0	53.5	49.0	44.5	41.5	37.3	34.0	32.3
P_{wb}, kips	6390	4900	3730	2780	2150	1610	1310	944	716	611
P_{fb}, kips	1440	1210	1000	832	684	583	483	398	334	298
L_p, ft	14.8	14.7	14.6	14.5	14.4	14.3	14.2	14.1	14.1	13.3
L_r, ft	110	100	91.6	83.4	76	70.1	64.5	58.9	54.7	49.6

Effective length KL (ft) with respect to least radius of gyration r_y

Effective Length

- The AISC LRFD table presented earlier presents values for the design load based on a slenderness ratio calculated using the minimum radius of gyration, r_y . Consider now the figure shown.



$KL = 13'$

(a) Minor Axis Buckling

$KL = 26'$

(b) Major Axis Buckling

Effective Length

- In such a case, slenderness about the minor axis may not control because the effective length for minor axis buckling is half that for major axis buckling. In this case, the effective slenderness ratio must be checked about each axis.
- The tables in Part 4 of the AISC specification can still be used but one must now check for the following two slenderness ratios:

$$\left(\frac{KL}{r}\right)_x \quad \text{and} \quad \left(\frac{KL}{r}\right)_y$$

Example Problems for Columns

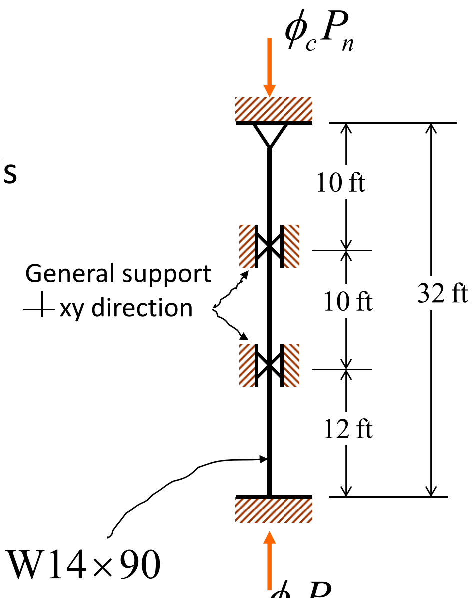
- Example 3
  a. Using AISC Manual, determine the design strength $\phi_c P_n$ of the 50 ksi axially loaded W14 × 90 shown in the figure. Because of its considerable length, this column is braced perpendicular to its weak axis at the points shown in the figure. These connections are assumed to permit rotation of the member in a plane parallel to the plane of the flanges. At the same time, however, they are assumed to prevent translation or sideway and twisting

Example Problems for Columns

- Example 3 (cont'd)

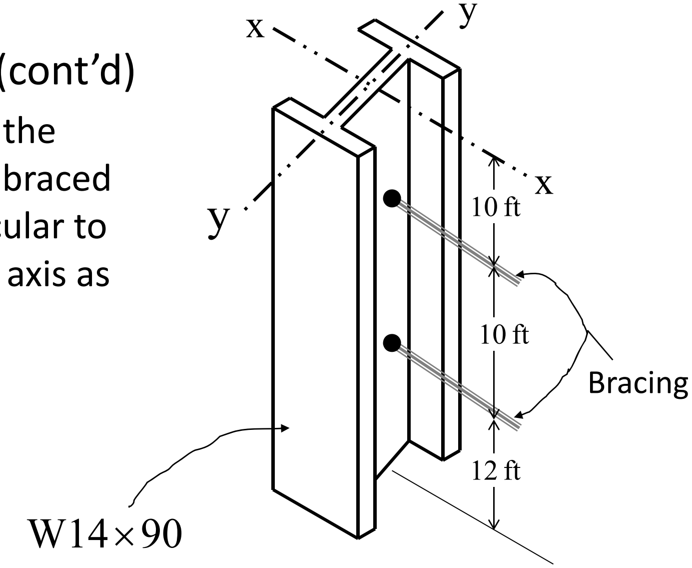
  of the cross section about a longitudinal axis passing through the shear center of the cross section.

  – Repeat part (a) using the column tables of Part 4 of the AISC Manual.



Example Problems for Columns

- Example 3 (cont'd)
  – Note that the column is braced perpendicular to its weak y axis as shown.



Example Problems for Columns

- Example 3 (cont'd)

  a. The following properties of the W14 × 90 can be obtained from the AISC Manual as

  $A = 26.5\ \text{in}^2 \qquad r_x = 6.14\ \text{in} \qquad r_y = 3.70\ \text{in}$

  <u>Determination of effective lengths:</u>

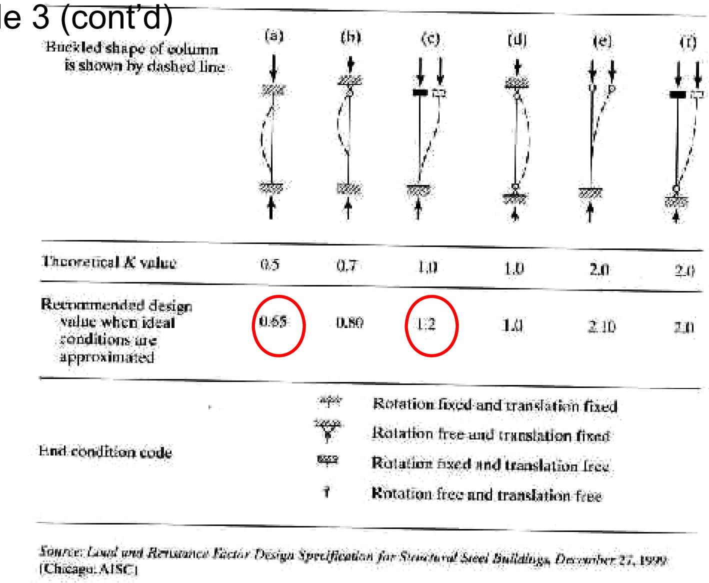
$$K_x L_x = (0.8)(32) = 25.6\ \text{ft}$$
$$K_y L_y = (1.0)(10) = 10\ \text{ft}$$ ← Governs for $K_y\ L_y$
$$K_x L_y = (0.8)(12) = 9.6\ \text{ft}$$

See Table for the *K* values

Example Problems for Columns

Example 3 (cont'd)

Table 1



Buckled shape of column is shown by dashed line	(a)	(b)	(c)	(d)	(e)	(f)
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0

End condition code:
- Rotation fixed and translation fixed
- Rotation free and translation fixed
- Rotation fixed and translation free
- Rotation free and translation free

Source: Load and Resistance Factor Design Specification for Structural Steel Buildings, December 27, 1999 (Chicago: AISC)

Example Problems for Columns

- Example 3 (cont'd)

Computations of slenderness ratios:

$$\left(\frac{KL}{r}\right)_x = \frac{12\times 25.6}{6.14} = 50.03 \longleftarrow \text{Governs}$$

$$\left(\frac{KL}{r}\right)_y = \frac{12\times 10}{3.70} = 32.43$$

Design Strength:

$$\frac{KL}{r} = 50.03 \approx 50, \text{Table 3-50 gives } \phi_c F_{cr} = 35.4 \text{ ksi}$$

$$\therefore \phi_c P_n = \phi_c F_{cr} A_g = 35.4(26.5) = 938 \text{ k}$$

Example Problems for Columns

- Example 3 (cont'd)

b. Using columns tables of Part 4 of AISC Manual:

Note: from part (a) solution, there are two different KL values:

$$K_x L_x = 25.6 \text{ ft and } K_y L_y = 10 \text{ ft}$$

Which value would control? This can accomplished as follows:

$$\frac{K_x L_x}{r_x} = \text{Equivalent } \frac{K_y L_y}{r_y}$$

Example Problems for Columns

- Example 3 (cont'd)

$$\text{Equivalent } K_y L_y = r_y \frac{K_x L_x}{r_x} = \frac{K_x L_x}{r_x / r_y}$$

The controlling $K_y\, L_y$ for use in the tables is larger of the real $K_y\, L_y$ = 10 ft, or equivalent $K_y\, L_y$:

$$\frac{r_x}{r_y} \text{ for W14}\times\text{90 from bottom of column tables} = 1.66$$

$$\text{Equivalent } K_y L_y = \frac{25.6}{1.66} = 15.43 > K_y L_y = 10 \text{ ft}$$

$$\text{For } K_y L_y = 15.42 \text{ and by interpolation:}$$

$$\phi_c P_n = 938 \text{ k}$$

Example Problems for Columns

- Example 3 (cont'd)

The Interpolation Process:

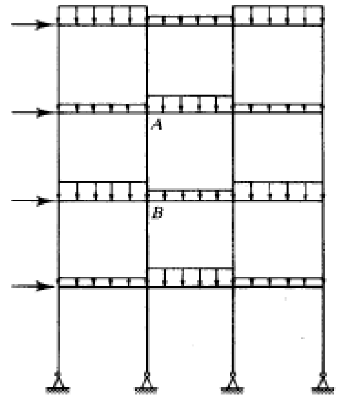
- For $K_y\, L_y$ = 15 ft and 16 ft, column table (P. 4-23) of Par 4 of the AISC Manual, gives respectively the following values for $\phi_c\, P_n$: 947 k and 925 k. Therefore, by interpolation:

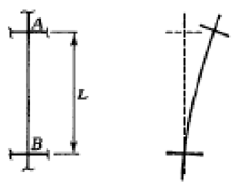
$$\begin{array}{ll} 15 & 947 \\ 15.42 & \phi_c P_n \\ 16 & 925 \end{array} \Rightarrow \frac{\phi_c P_n - 947}{925 - 947} = \frac{15.42 - 15}{16 - 15} \Rightarrow \phi_c P_n = 938 \text{ k}$$

Effective Length

For columns in moment-resisting frames, the tabulated values of K presented on Table C-C2.1 of AISC Specification will not suffice for design. Consider the moment-frame shown that is permitted to sway.

- Columns neither pinned not fixed.
- Columns permitted to sway.
- Columns restrained by members framing into the joint at each end of the column





Effective Length

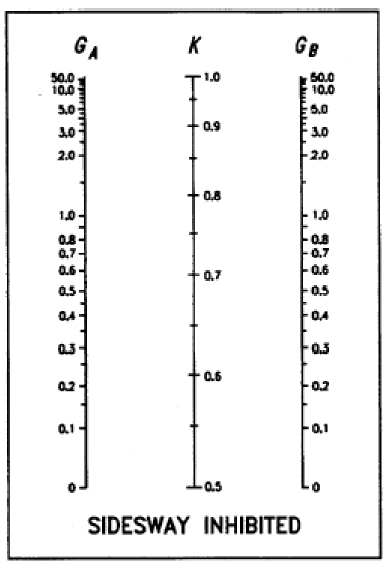
The effective length factor for a column along a selected axis can be calculated using simple formulae and a nomograph. The procedure is as follows:

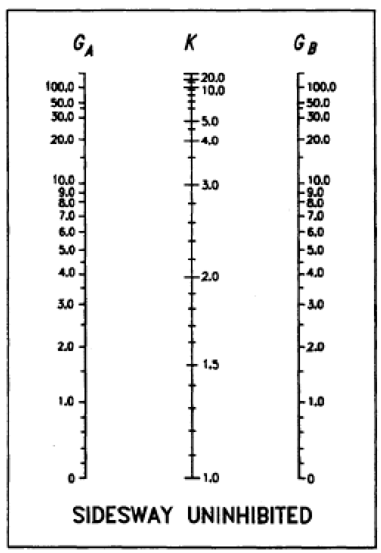
- Compute a value of G, defined below, for each end of the column, and denote the values as G_A and G_B , respectively

$$G = \frac{\Sigma(EI/L)_{col}}{\Sigma(EI/L)_{beam}}$$

- Use the nomograph provided by AISC (and reproduced on the following pages). Interpolate between the calculated values of G_A and G_B to determine K

Effective Length

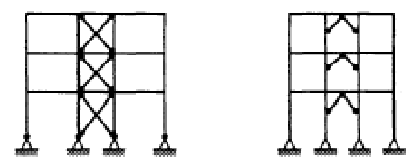




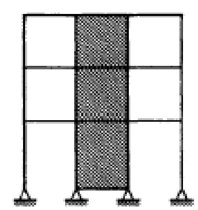
AISC specifies G = 10 for a pinned support and G = 1.0 for a fixed support.

Effective Length

- The distinction between braced (sidesway inhibited) and unbraced (sidesway inhibited) frames is important, as evinced by difference between the values of K calculated above.
- What are bracing elements?



(a) Diagonal bracing



(b) Shear Walls
(masonry, reinforced concrete, or steel plate)

Beam-Columns

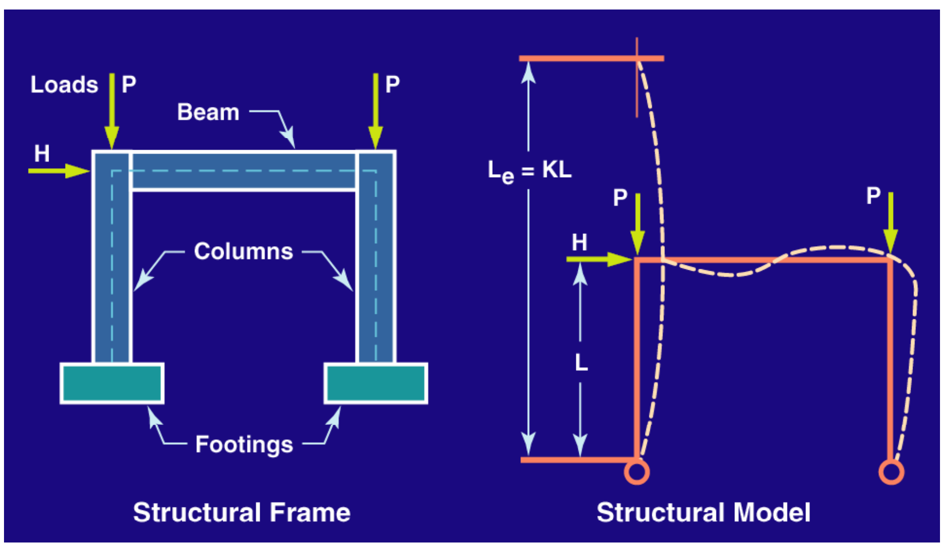
Based on the nature of loading of a column, columns may be classified as beam-columns. Such columns support both lateral and axial loading.

Loads and Member Forces

The following column loading and effects should be determined:

- Bending and axial loading
- Eccentricity of applied load
- Shear loading

Bending and Axial Load on a Column



Beam-Columns

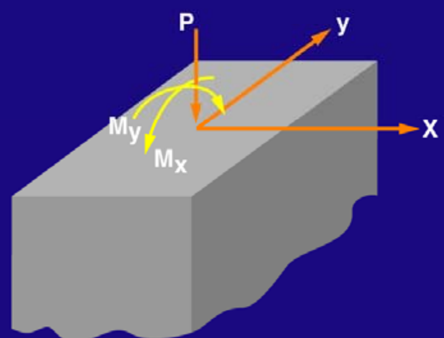
The basic stresses in a structural member due to axial load, P, and bending, M, and their applicable formulas are:

- **Axial stress,** $f_a = \frac{P}{A}$
- **Bending (flexural) stress,** $f_b = \frac{M}{S}$
- **Combined stress,** $f = f_a \pm f_b = \frac{P}{A} \pm \frac{M}{S} \leq F_a = \frac{F_y}{F.S.}$

Axial Load and Bending

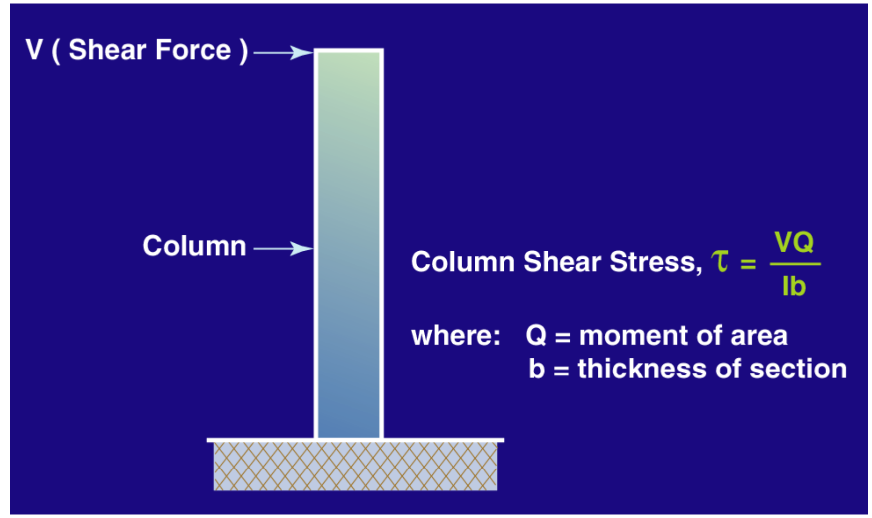
Axial load and bending about both axes:

Interaction formula: $$\frac{f_a}{F_b} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \le 1$$

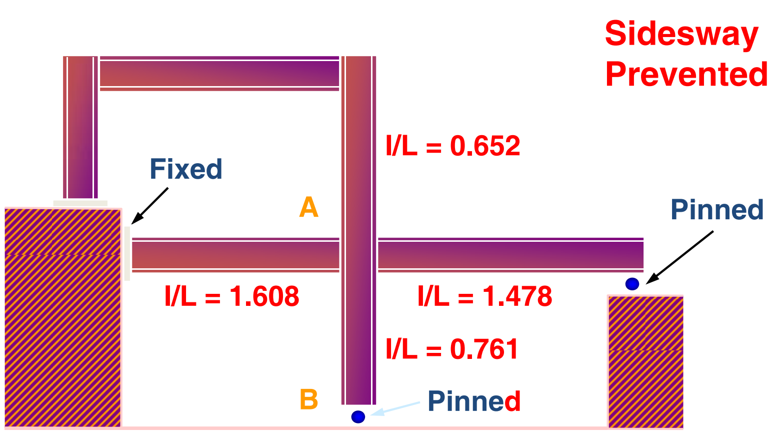


Calculating Shear Loading

For columns subject to shear loading:



Sample Problem: Determining K Factors for Columns



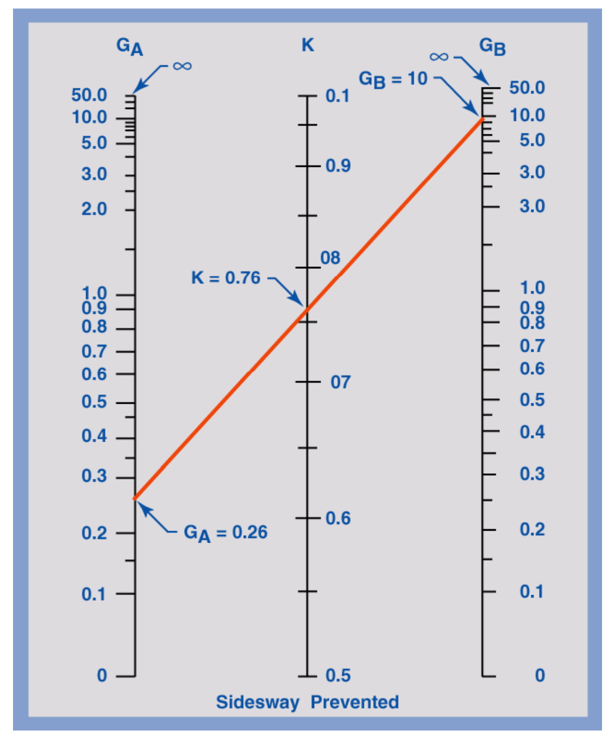
Effective Length Factor K in Column Design

$$G_A = \frac{0.652 + 0.761}{2(1.608) + 1.5(1.478)}$$

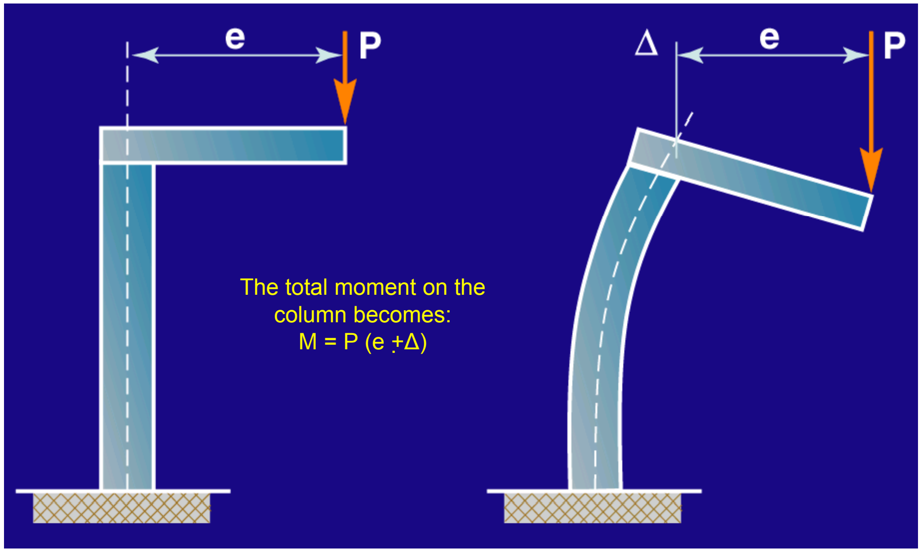
$$= 0.260$$

$$G_B = 10$$

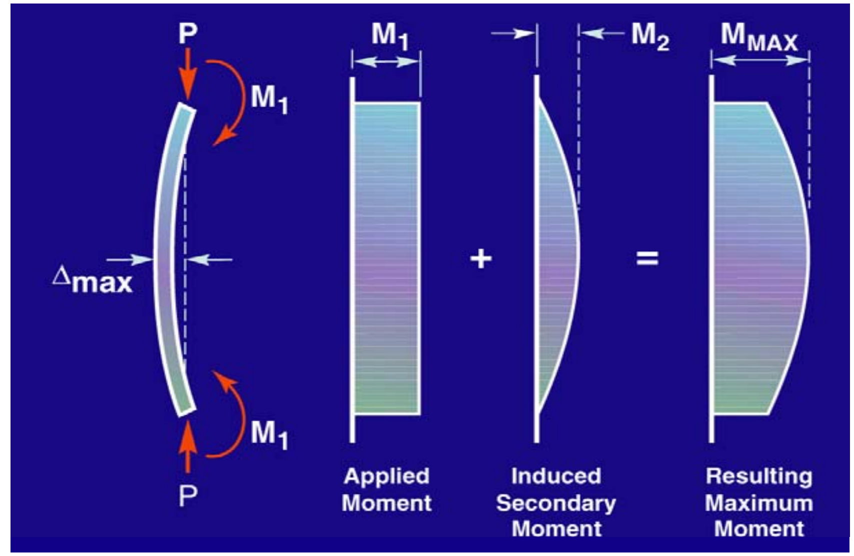
Answer: K = 0.76



Sideway Effect



Moment Magnification Effects



Since: $M_{max} = M_1 + M_2$

Then: $M_{max} = M_1 + P\Delta_{max}$

Combined Bending and Axial Load

- Doubly and Singly Symmetric Members in Flexure and Compression
  - $For\ \dfrac{P_u}{\phi_c P_n} \geq 0.2$ $\qquad \dfrac{P_u}{\phi_c P_n} + \dfrac{8}{9}\left(\dfrac{M_{ux}}{\phi_b M_{nx}} + \dfrac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$ (H1-1a)
  - $For\ \dfrac{P_u}{\phi_c P_n} < 0.2$ $\qquad \dfrac{P_u}{2\phi_c P_n} + \left(\dfrac{M_{ux}}{\phi_b M_{nx}} + \dfrac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$ (H1-1b)
- Unsymmetric and Other Members in Flexure and Compression
  - $\left|\dfrac{f_a}{F_a} + \dfrac{f_{bw}}{F_{bw}} + \dfrac{f_{bz}}{F_{bz}}\right| \leq 1.0$ (H2-1)

Methods of Second-order Analysis

- Amplified First–Order Elastic Analysis (Section C2.1b)

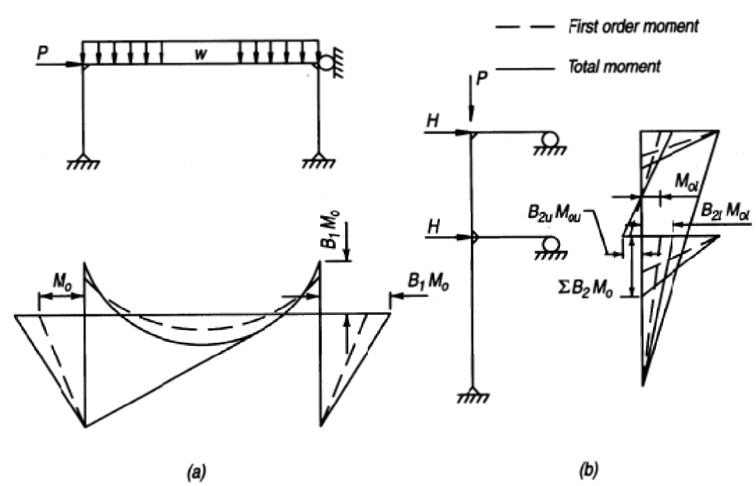


Fig. C-C2.1. Moment amplification.

2nd-Order Analysis by Amplified 1st-Order Elastic Analysis

- 2^{nd}-order flexural strength M_r

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(C2-1a)}$$

- 2nd-order axial strength P_r

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(C2-1b)}$$

where

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \qquad \text{(C2-2)}$$

$$B_2 = \frac{1}{1 - \alpha \Sigma P_{nt} / \Sigma P_{e2}} \geq 1 \qquad \text{(C2-3)}$$

2nd-Order Analysis by Amplified 1st-Order Elastic Analysis

- B_1 is an amplifier to account for second order effects caused by displacement between brace points (P-δ)
- B_2 is an amplifier to account for second order effects caused by displacements of braced points (P-Δ)
- If $B_1 \leq 1.05$, it is conservative that $M_r = B_2(M_{nt} + M_{lt})$

2nd-Order Analysis by Amplified 1st-Order Elastic Analysis

- C_m is a coefficient assuming no lateral translation of frame (no transverse loading)

$$C_m = 0.6 - 0.4(M_1 / M_2) \qquad \text{(C2-4)}$$

- P_{e1} is the elastic critical buckling resistance with zero sidesway

$$P_{e1} = \frac{\Pi^2 EI}{(K_1 L)^2} \qquad \text{(C2-5)}$$

- ΣP_{e2} is the elastic critical buckling resistance for the story
  - For moment frames (C2-6a)

$$\Sigma P_{e2} = \Sigma \frac{\Pi^2 EI}{(K_2 L)^2}$$

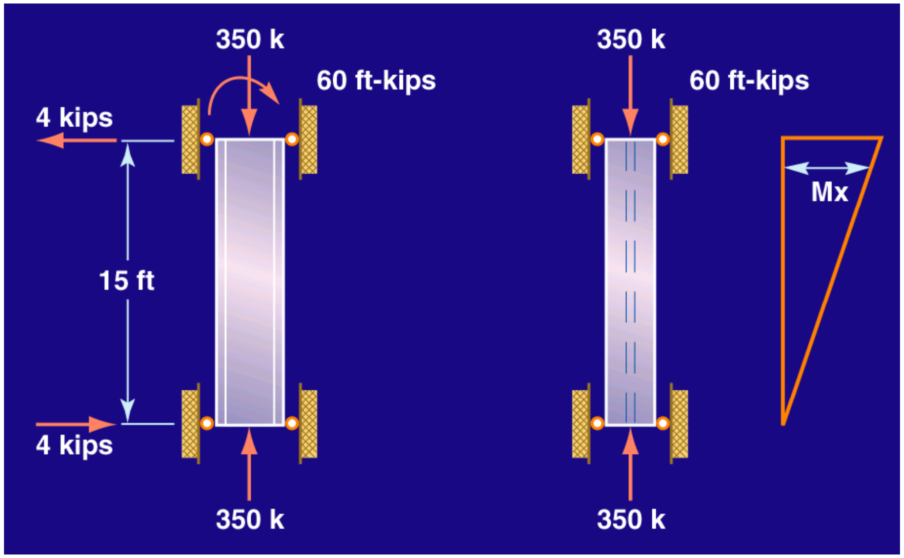
  - For all types (C2-6b)

$$\Sigma P_{e2} = R_M \frac{\Sigma HL}{\Delta_H}$$

Sample Problem: Determining Allowable Axial Compressive Stress of a Column

Refer to AISC Manual of Steel Construction, 13th edition, Part 4, to determine the allowable axial compressive stress for a column with an effective length of 12 ft and a radius of gyration of 1.49 in. F_y = 36 ksi steel.

Sample Problem: Designing Column with Combined Axial and Bending Loads



Design Example of Compression Members

- For W10×33, calculate the available axial strength
  For a pinned-pinned condition, K = 1.0
  Since $KL_x = KL_y = 14.0$ ft and $r_x > r_y$, the y-y axis will govern.
  $P_c = \varphi_c P_n = 253$ *kips*

- Calculate the required flexural strengths including second order amplification ($C_m = 1.0$ & $\alpha = 1.0$)

$$P_{e1} = \frac{\Pi^2 EI}{(K_1 L)^2} \qquad P_{e1} = \pi^2(29000)(171\ in^4)/(1 \times 14 \times 12)^2 = 1730\ kips$$

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \qquad B_1 = 1/[1 - 1.0(350/1730)] = 1.254$$

Amplified $M_{ux} = B_1 (M_{ux}) = 1.254\ (60) = 75.24$ *ft-kips*

6300. Design -

6310. Structural Steel Members and Components

Objective and Scope Met

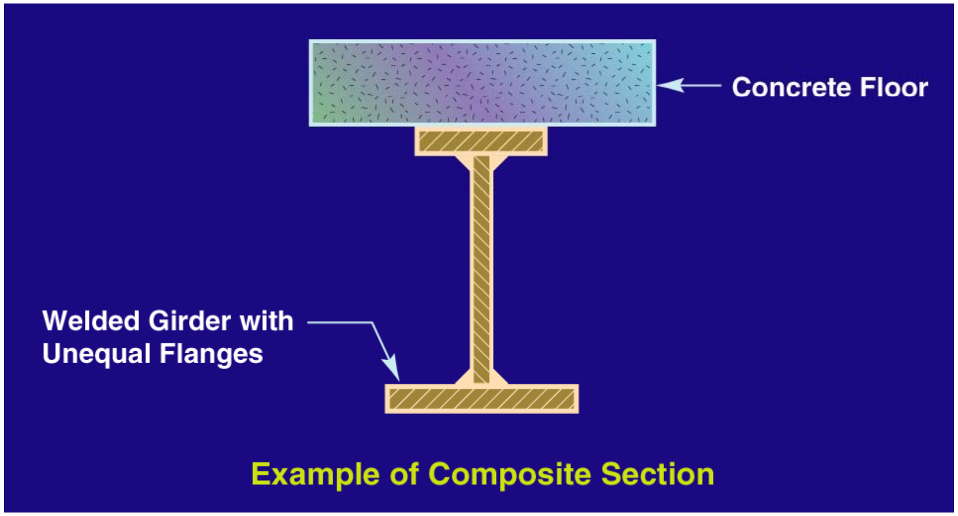
- Module 3: Compression
  - Introduction
  - Design factors
  - Load and member forces
  - Stability and end-support considerations
  - AISC-allowable stress and load tables
  - Parameters and format of column design tables
  - Design examples of columns

6310. Structural Steel Members and Components – Module 4: Composite Members

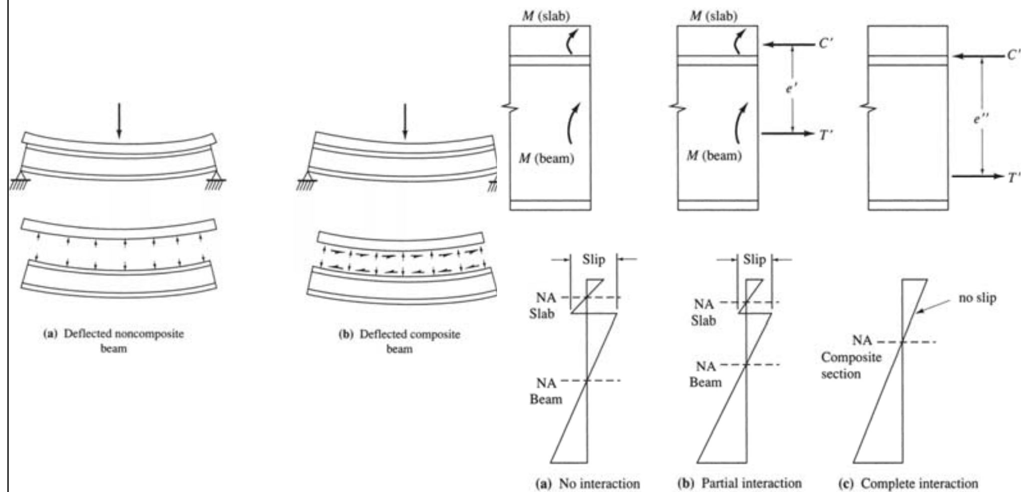
This section of the module covers:

- Composite Action
- Effective Width
- Nominal Moment Strength
- Shear Connectors, Strength and Fatigue
- Formed Steel Deck
- Composite Column

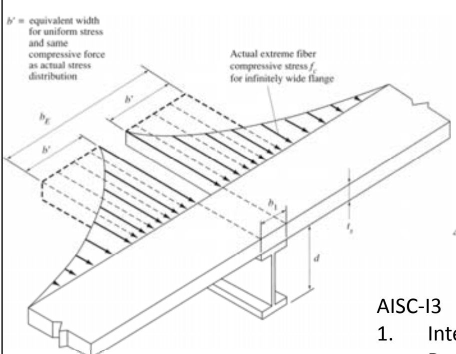
Calculating Composite Beam Section Properties

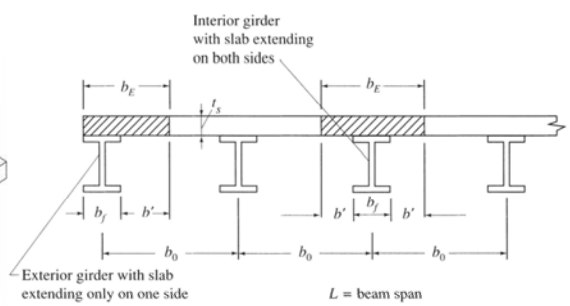


Composite Action



Effective Width





AISC-I3

1. Interior
   - $B_E \le L/4$
   - $B_E \le b_0$ (for equal beam spacing)
2. Exterior
   - $B_E \le L/8$ + (dist from beam center to edge of slab
   - $B_E \le b_0/2$ + (dist from beam center to edge of slab)

Nominal Moment Strength

Nominal Moment Strength of Fully Composite Section
(AISC 13th Edition Art. I3.2a)

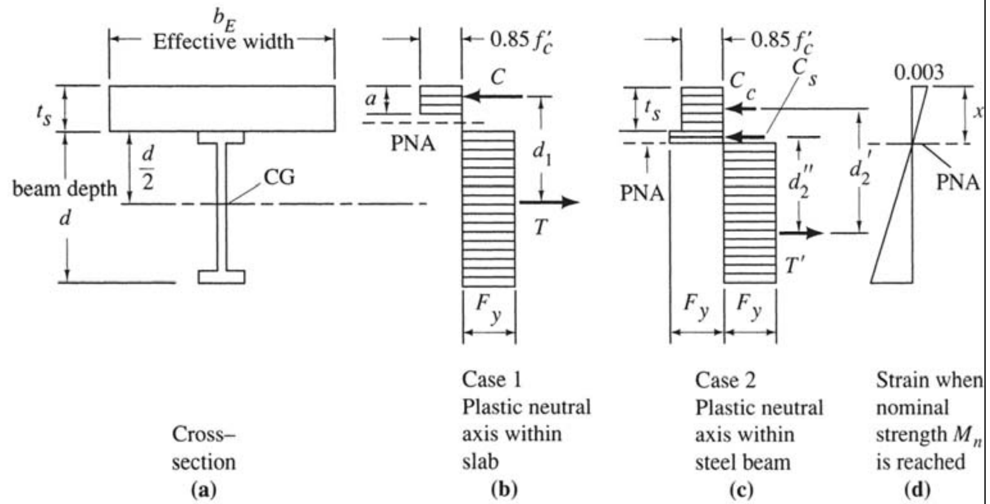
1.
$$h_c / t_w \le \left(\lambda_p = 3.76 / \sqrt{E/F_{yf}} \right)$$

M_n = based on <u>plastic stress distribution</u> on the Composite Section;
$\Phi_b = 0.9$

2.
$$h_c / t_w > \left(\lambda_p = 3.76 / \sqrt{E/F_{yf}} \right)$$

M_n = based on <u>superposition of elastic stresses</u>, considering the effect of shoring;
$\Phi_b = 0.9$

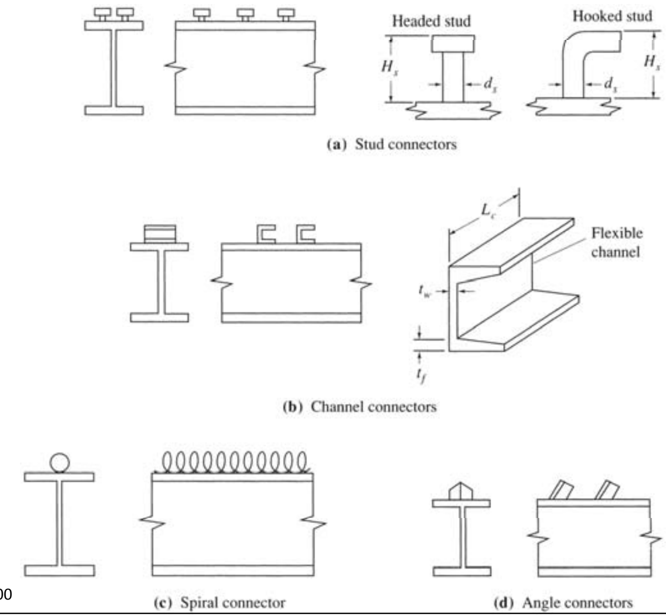
Plastic Stress Distribution



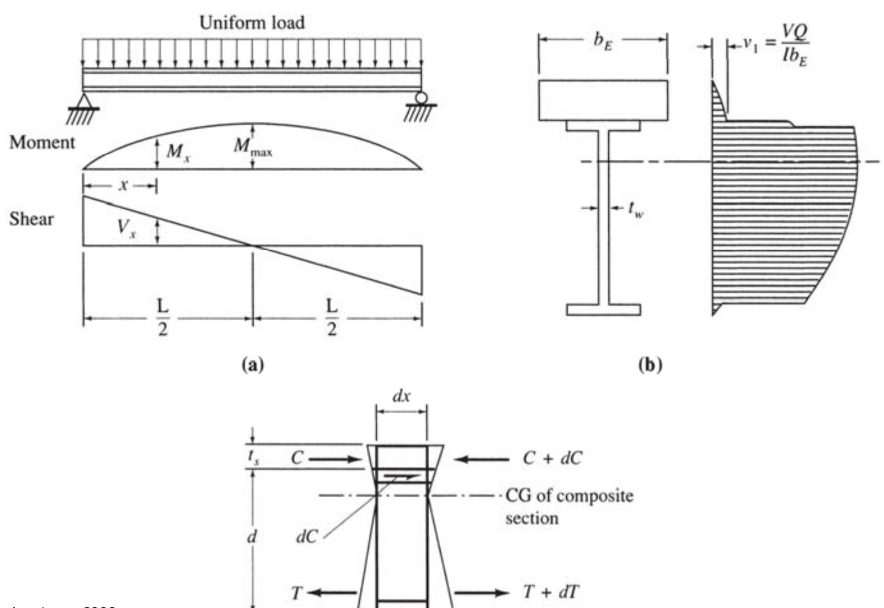
Case 1 (if $a \le t_s$)

Case 2 (if $a > t_s$)

Shear Connectors



Shear Variation



Nominal Strength Q_n

Q_n =

1. Headed Steel Stud (AISC Eq. I3-3)

$$Q_n = 0.5 A_w \sqrt{f_c' E_c} \le R_g R_p A_{sc} F_u$$

2. Channel Connectors (AISC Eq. I3-4)

$$Q_n = 0.3(t_f + 0.5 t_w) L_c \sqrt{f_c' E_c}$$

Condition	R_g	R_p
No decking*	1.0	1.0
Decking oriented parallel to the steel shape		
$\frac{w_r}{h_r} \ge 1.5$	1.0	0.75
$\frac{w_r}{h_r} < 1.5$	0.85**	0.75
Decking oriented perpendicular to the steel shape		
Number of studs occupying the same decking rib		
1	1.0	0.6+
2	0.85	0.6+
3 or more	0.7	0.6+

h_r = nominal rib height, in. (mm)

w_r = average width of concrete rib or haunch (as defined in Section I3.2c), in. (mm)

Nominal Strength Q_n

TABLE 16.8.1 Nominal Strength Q_n (kips) for Stud and Channel Shear Connectors Used with No Decking ($R_g = R_p = 1.0$) and Normal-Weight Concrete†

Connector	Concrete strength f'_c (ksi) 3.0	3.5	4.0
1/2″ diam × 2″ headed stud	9.4	10.5	11.6
5/8″ diam × 2-1/2″ headed stud	14.6	16.4	18.1
3/4″ diam × 3″ headed stud	21.0	23.6	26.1
7/8″ diam × 3-1/2″ headed stud	28.6	32.1	35.5
Channel C3×4.1	$10.2L_c$*	$11.5L_c$	$12.7L_c$
Channel C4×5.4	$11.1L_c$	$12.4L_c$	$13.8L_c$
Channel C5×6.7	$11.9L_c$	$13.3L_c$	$14.7L_c$

†AISC Formula (I3-3), Eq. 16.8.5, used for studs and AISC Formula (I3–4), Eq. 16.8.6, used for channels. Studs, A108 Type 2, $F_u^b = 60$ ksi.

*L_c = Length of channel, in.

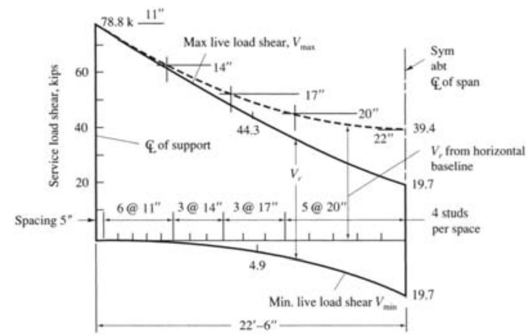
Connector Design – Fatigue Strength

$$p \le \frac{nZ_r I}{V_{sr} Q}$$ (AASHTO LRFD Eq. 6.10.7.4.1b-1)

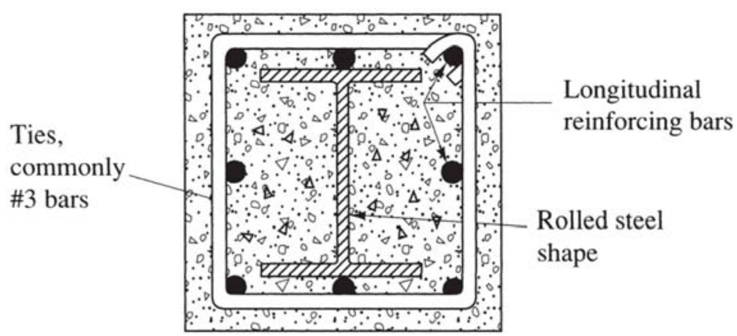
$Z_r = \alpha\, d^2 \ge 5.5\, d^2/2$; (AASHTO LRFD Eq. 6.10.7.4.2-1)

where $\alpha = 34.5 - 4.28 \log N$ (AASHTO LRFD Eq. 6.10.7.4.2-2)

Example:



Composite Column Section (rolled steel shape encased in concrete)



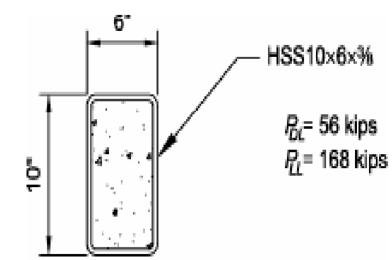
Using Effective Section Properties

$$P_0 = A_s F_y + A_{sr} F_{yr} + 0.85 A_c f'_c$$

$$P_{e1} = \frac{\Pi^2 EI_{eff}}{(K_1 L)^2} \qquad EI_{eff} = E_s I_s + 0.5 E_s I_{se} + C_1 E_c I_c$$

Filled Composite Column Example

Determine if a 14-ft long HSS10×6×⅜ ASTM A500 grade B column filled with $f'_c = 5$ ksi normal weight concrete can support a dead load of 56 kips and a live load of 168 kips in axial compression. The column is pinned at both ends and the concrete at the base bears directly on the base plate. At the top, the load is transferred to the concrete in direct bearing.



Filled Composite Column Example

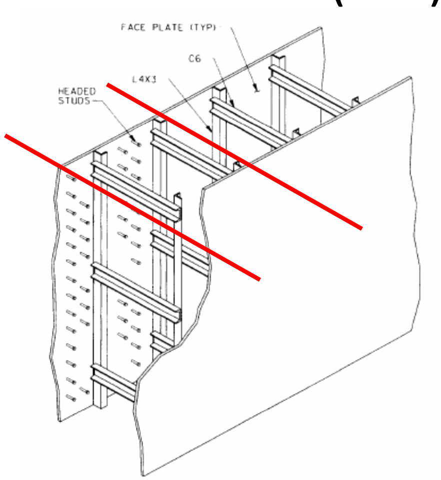
- $A_c = b_f h_f + \pi(r-t)^2 + 2b_f(r-t) + 2h_f(r-t)$
  $A_c = (8.5\text{ in.})(4.5\text{ in.}) + \pi(0.375\text{ in.})^2 + (8.5\text{ in.})(0.375\text{ in.}) + 2(4.5\text{ in.})(0.375\text{ in.}) = 48.4\text{ in.}^2$
- $$I_c = \frac{b_1 h_1^2}{12} + \frac{2(b_2)(h_2^2)}{12} + 2(r-t)\left(\frac{\pi}{8} - \frac{8}{9\pi}\right) + 2\left(\frac{\pi(r-t)^2}{2}\right)\left(\frac{h_2}{2} - \frac{4(r-t)}{3\pi}\right)^2 = 111 in.^4$$
- $$P_0 = A_s F_y + A_{sr} F_{yr} + 0.85 A_c f'_c$$
- $P_0 = (10.4\text{ in.}^2)(46\text{ksi}) + 0.85(48.4\text{ in.}^2)(5\text{ ksi}) = 684\text{kips}$
- $$EI_{eff} = E_s I_s + 0.5 E_s I_{se} + C_3 E_c I_c$$
- $EI_{eff} = (29{,}000\text{ kis})(61.8\text{ in.}^4) + (0.90)(3{,}900\text{ ksi})(111\text{ in.}^4)$
  $= 2{,}180{,}000\text{ kip-in.}^2$

Filled Composite Column Example

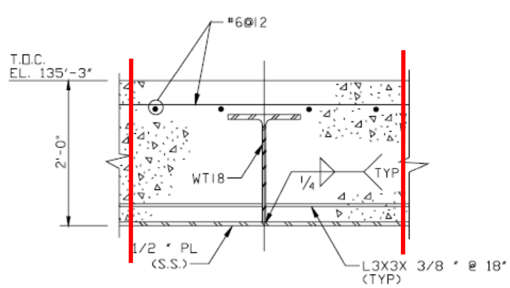
$$P_{e1} = \frac{\Pi^2 EI_{eff}}{(K_1 L)^2}$$

- $P_e = \pi^2(2{,}180{,}000\text{ kip-in.}^2)/(1.0(14\text{ ft})(12\text{ in./ft}))^2 = 762\text{ kips}$
- $P_0/P_e = 684\text{ kips}/762\text{ kips} = 0.898 \le 2.25$
- $$P_n = P_0\left[0.658^{p_0/P_e}\right] = (684 kips)\left[0.658^{0.898}\right] = 470 kips$$
- $\varphi_c P_n = 0.75(470\text{ kips}) = 353\text{ kips} > 336\text{ kips}$ o.k.

AP1000 Sandwich Steel-Concrete-Steel (SCS) Structures



Typical Structural Wall Module



Typical Structural Floor Module

AP1000 Sandwich Steel-Concrete-Steel (SCS) Structures

How the composite section works:

- Composite action is between the concrete and the steel faceplates.
- The steel plates and the concrete act as a composite section after the concrete has reached sufficient strength
- The composite section resists bending moment by one face resisting tension and the other face resisting compression
- The steel plate resists the tension and behaves as reinforcing steel in reinforced concrete
- The composite section is under-reinforced so that the steel would yield before the concrete reaches its strain limit of 0.003 in/in
- The steel faceplates are strained beyond yield to allow the composite section to attain its ultimate capacity

AP1000 Sandwich Steel-Concrete-Steel (SCS) Structures

Design:

- Design theory is the same as earlier described for concrete-filled tube section for compression and composite beam for flexure
- The size and spacing of the shear studs is based on Section Q1.11.4 of AISC-N690 to develop full

Advantages:

- Based on research, concrete and steel composites similar to the structural modules have significant advantages over reinforced concrete elements of equivalent thickness and reinforcement ratios:
- Over 50 percent higher ultimate load carrying capacity
- Three times higher ductility
- Less stiffness degradation under peak cyclic loads, 30 percent for concrete and steel composites versus 65 percent for reinforced concrete

6300. Design -

6310. Structural Steel Members and Components

Objective and Scope Met

- Module 4: Composite Members
  - Composite Action
  - Effective Width
  - Nominal Moment Strength
  - Shear Connectors, Strength and Fatigue
  - Formed Steel Deck
  - Composite Column

6300. Design -

6310. Structural Steel Members and Components

Objective and Scope Met

- Module 1: Tension
- Module 2: Flexure and Shear
- Module 3: Compression
- Module 4: Composite Members